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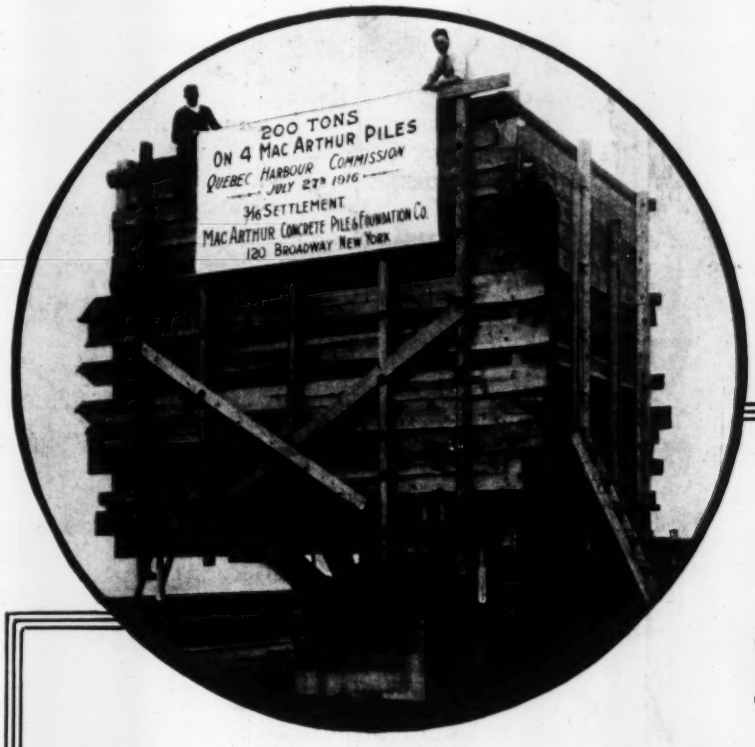
Even in Treacherous Soil



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Engineering News

VOLUME 76 SEPTEMBER 14, 1916 NUMBER 11

Construction and Maintenance of Roads in the Arid West

SYNOPSIS—A traveler's notes on road-building materials and road construction under adverse climatic conditions. The problem of gravel-road maintenance under arid conditions and various attempts to solve it.

A great section of country lying in the western part of Texas, Arizona, New Mexico and Nevada was for many years known as the Great American Desert, and the greater part of it still is an arid and desert country. Notwithstanding the fact that counties in these arid regions are as large as states in the East and that the population of the counties, concentrated in the irrigated valleys, is about that of townships in the Central West there is great activity in road building, and the people are keenly appreciative of improved roads.

It is a country where practically every property owner has one or more motor cars, and where freighting heavy loads for long distances on the roads by team or motor is a necessary practice. The traffic in vehicles per day on most of the roads would seem insignificant compared with traffic records on roads in the more thickly settled parts of the country; but owing to the dry atmosphere and scanty rainfall the destructive effects of any traffic on dirt or gravel roads are magnified many times over what they would be in a country where the roads could be maintained by established methods. Anything but an improved dirt or gravel road, costing from $500 to $2,500 per mi., is obviously out of the question except in very rare instances.

Texas has no state highway department, but much road improvement work has been done by the counties. At Dallas, J. F. Witt, County Engineer, has about 500 mi. of gravel or hard surface road under his jurisdiction. Climatic conditions here are of course not so aggravating as in the western part of the state, yet the long, hot, dry summers give an introduction to the conditions farther west. During this dry season the dust nuisance becomes so acute that not only are gravel-road surfaces rapidly disintegrated, but there is no pleasure or comfort in riding over them.

The use of calcium chloride has not proved of any benefit. It is evident that to be effective this material must be used in a more humid climate where an appreciable amount of water vapor is present in the atmosphere.

FIG. 1. AN UNIMPROVED ROAD IN IRRIGATED VALLEY, NEW MEXICO

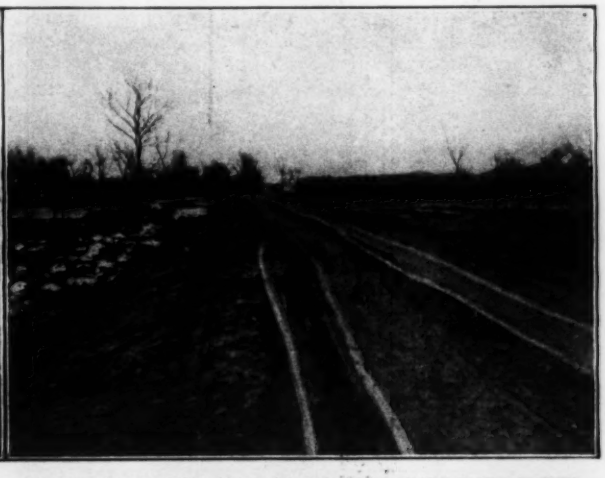

FIG. 2. RELOCATION OF SAME ROAD IMPROVED WITH SAND-CLAY SURFACING

Mr. Witt has also had poor success with the light semi-asphaltic or nonasphaltic oils. In a dry climate these are not sufficient to hold the dust by capillary attraction, and clouds of dust particles coated with grease are obviously more objectionable than dry dust. On gravel roads in good condition oil containing 50% asphalt has given satisfaction in Dallas County.

Asphaltic oil coating, costing $500 to $1,500 per mi., is obviously too expensive for the ordinary country roads in this section of the country. For $150 per mi. per yr. well-built gravel roads have been maintained in fair condition by periodically regrading and planing them with a road grader to remove ruts and using new material for patching where necessary. This answers for the first six or seven years after construction, then it becomes necessary to entirely resurface the road with new gravel

which, owing to the already graded and compacted subgrade, may be done ordinarily for about half the first cost of the road.

For the more heavily traveled roads oiled gravel surfacing on a crushed-limestone base has given fair satisfaction and is about the cheapest form of bituminous-road construction for this section. Farther south, in Harris County, oyster shells have been used experimentally for the foundation course of gravel roads with apparently excellent results. These shells are not hard enough to withstand any considerable amount of wear, yet they mat and bind together to form an excellent base for any type of hard surfacing.

The experience with shell-surfaced roads in Texas has been unsatisfactory. A small amount of steel-tired traffic wears the surface very rapidly and motor traffic raises great clouds of suffocating dust. Oil has been tried, but owing to the presence of the dust and to the fact that the shells mat together so compactly, practically no penetration of the oil is possible. Shell roads have, however, been resurfaced with gravel by first harrowing the old surface and adding about 2 in. or less of new shells and

FIG. 3. MAIN STREET OF CHEYENNE, WYO., "PAVED"

2 in. of fine, washed gravel. The whole is then reharrowed and compacted, giving a much more satisfactory surface than the shell alone. A 16-ft. road of this type costs from $2,800 to $3,500 per mi.

In Bexar County (Fred Bennett, County Engineer), of which San Antonio is the county seat, nearly three-quarters of a million dollars has been expended on graveling and improving roads during the last two or three years. Particular attention is paid to maintenance, which is complicated by the arid conditions typical of the country from here west. On these roads tar oils have proved particularly satisfactory. The hardening and cracking of tarred surfaces two or three years old, common in the North, is apparently prevented or delayed by the near-tropical climatic conditions.

Two kinds of gravel are used in Bexar County; in the northern part a good flint gravel, in the southern part adobe gravel. The former makes the most durable road, but the latter is the kind most used throughout the Southwest. Adobe gravel forms a hard, compact road surface, but dusts and ruts badly in the dry atmosphere. No oil penetrates a compacted surface of this material more than ½ in.

The Bexar County roads probably have been constructed with as much care and engineering supervision as those in any of the far western states. Particular attention has been paid to gravel inspection. Men employed for this purpose are paid $80 per month. The roads are rolled with 12- or 15-ton road rollers—an unusual refinement for this section. The county is divided into precincts and a regular allowance of $5,000 per precinct is made for maintenance.

In El Paso County bituminous macadam and asphaltic concrete roads have been tried, and with very poor success. At present several miles of concrete pavement is being laid on county roads—probably the only use of this material in western Texas, or in that part of the Southwest. There are cement mills at El Paso, which was probably one of the deciding factors in the choice of material.

The majority of Texas gravels are of limestone origin and are not so suitable for roads as the granite gravels found farther west.

Gravel Roads in New Mexico

New Mexico has a state highway department (James A. French, State Engineer) and systematic improvement of roads is in progress. The annual rainfall in this state varies from 8 in. in the lower valleys and plains to 25 and 30 in. in the higher mountain regions. The precipitation in the mountains is mostly in the form of snow. Systematic dragging of the roads is impossible.

Topographically the state may be divided into river valleys, mesas and mountains. The mountains and mesas furnish plenty of material for cheap road building. The mesas are generally formed of deposits of clay and gravel and furnish ideal material. The areas in the river valleys are not so fortunate. The soil there is of three types—sandy stretches, river soil or adobe and the black soil of irrigated tracts.

Roads through the sandy and adobe soils are made mostly of the sand-clay type; in the first case adding clay and sand and in the second only sand or gravel, leaving the mixing and compacting to be done by traffic. In both instances it is possible to obtain fair roads at very small cost, but it is of course difficult to maintain them in good condition. In Valencia County, a few miles south of Albuquerque, A. E. St. Morris, County Highway Director, has improved many miles of road in this manner, in some instances with a wearing surface no more than 3 or 4 in. thick. In irrigated sections gravel roads are constructed more scientifically, being graveled, wetted and rolled. They are also being maintained to a certain extent by dragging, as these lowland roads are not infrequently in a saturated condition owing to retaining rain water and to seepage from adjoining irrigated lands (see Figs. 1 and 2).

Road Building in Arizona

The improvement of roads in Arizona is under a kind of dual control. State roads are built under the direction of state division engineers by convict and day labor, while the counties may sell bonds and build roads by contract or day labor as they see fit under the state highway department, the county engineers or through outside con-

FIG. 4. CURB AND GUTTER DETAIL, CHEYENNE GRAVEL PAVEMENT

sulting engineers. The topographic conditions are somewhat similar to New Mexico, but possibly more varied. The southern counties are near-tropical while the northern counties in the mountains have plenty of frost and snow.

Pima County (of which Tucson is the county seat) is now building 26 mi. of water-bound macadam, using local crushed lava rock known as malapi and screenings. This is the first of several miles of bond-issue roads in the county to be improved in this manner, probably the first waterbound macadam in any of the arid states. In some instances in Pima County water will be so rare that it will be necessary to roll the macadam dry and wait until the rainy season for the final rolling and finishing. Bids for the first 26 mi. of these roads were published in the Construction News supplement of *Engineering News,* July 6, 1916. Wm. H. Rosecrans & Co., of Chicago, are consulting engineers in charge.

A common road-building material in southern Arizona is a conglomerate known locally as caliche. It consists of gravel and fragments of stone cemented by lime. It is spread on the road in the same manner as gravel and under traffic forms a very hard, smooth surface, but in the long dry spells under a small amount of automobile traffic it rapidly loses large amounts of dust binder and ruts badly. A special method of constructing caliche roads in Maricopa County by use of irrigation water was described in *Engineering News,* May 4, 1916. The repair and maintenance of caliche roads, as of gravel roads in this section, consist merely in replacing the material which blows away as dust with fresh material. State Engineer Lamar Cobb is experimenting with asphaltic oils on caliche roads, and a small experimental stretch of Tarvia has been constructed in Maricopa County.

Yuma County in the extreme southwest corner of the state is spending several hundred thousand dollars in the construction of bitulithic and warrenite (asphaltic concrete) pavements. In the northern counties good gravel is available in most instances.

The best surfaced roads in Colorado are those where a disintegrated granite gravel has been used. This material is found in many parts of the Rocky Mountain states, but that in El Paso County, around Colorado Springs, is about the best. A surfacing of 6 in. of this material, which is like a very coarse sand with loamy binder, will retain a smooth, hard surface for two or three years under a traffic of as many as 50 to 100 motor cars per day.

The dull-red, disintegrated granite found on the line of the Union Pacific R.R. west of Cheyenne, Wyo., known as Sherman gravel, is familiar to engineers who have crossed the continent by that route, for it is used for ballasting many miles of the Union Pacific on either side of Cheyenne. It is an ideal road-surfacing material. The streets of Cheyenne, a city of about 12,000, are surfaced with it, with results almost equal to those of paving. When adequately sprinkled to avoid the dust nuisance this street surfacing seems to be quite satisfactory. Figs. 3, 4 and 5 show some of the Cheyenne streets.

All parts of Wyoming, however, are not so well supplied with road-building material. In the northeastern and central parts of the state black loam and gumbo soils are found and roads in these sections are almost impassable after continued rains. In the southwestern part of the state the soil is largely sandy loam and the roads are in bad condition in rainy weather. Dragging the roads in a country where the population averages 1.5 to the square mile is obviously impossible, so that little advantage is taken of the rainy spells.

Road Building and Maintenance in Utah

The State Highway Department of Utah has built a few miles of concrete pavement and with very fair success, but the great part of the state roads are merely graded, or graded and graveled. A method of rapid grading used on these roads was described in *Engineering News,* July 6, 1916.

Utah is one of the few states, possibly the only one, which has tried (experimentally) systematic sprinkling with water carts or trucks as a maintenance measure on gravel roads. Special reservoirs and water stations have been constructed for this purpose in Davis County. It is stated, however, by E. R. Morgan, State Road Engineer, that ordinary sprinkling has proved ineffective, and he proposes during the present year to try out a method of maintenance by which the road will receive a thorough soaking and then a dragging; in other words, he proposes to maintain gravel roads as they are maintained under more favorable climatic conditions by mak-

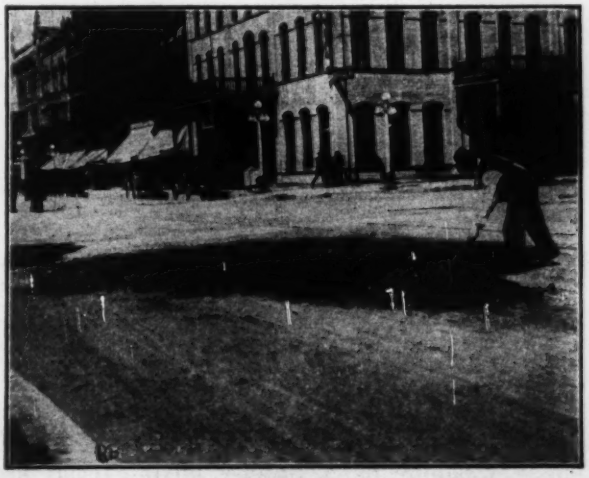

FIG. 5. "REPAIRING" CHEYENNE PAVEMENTS

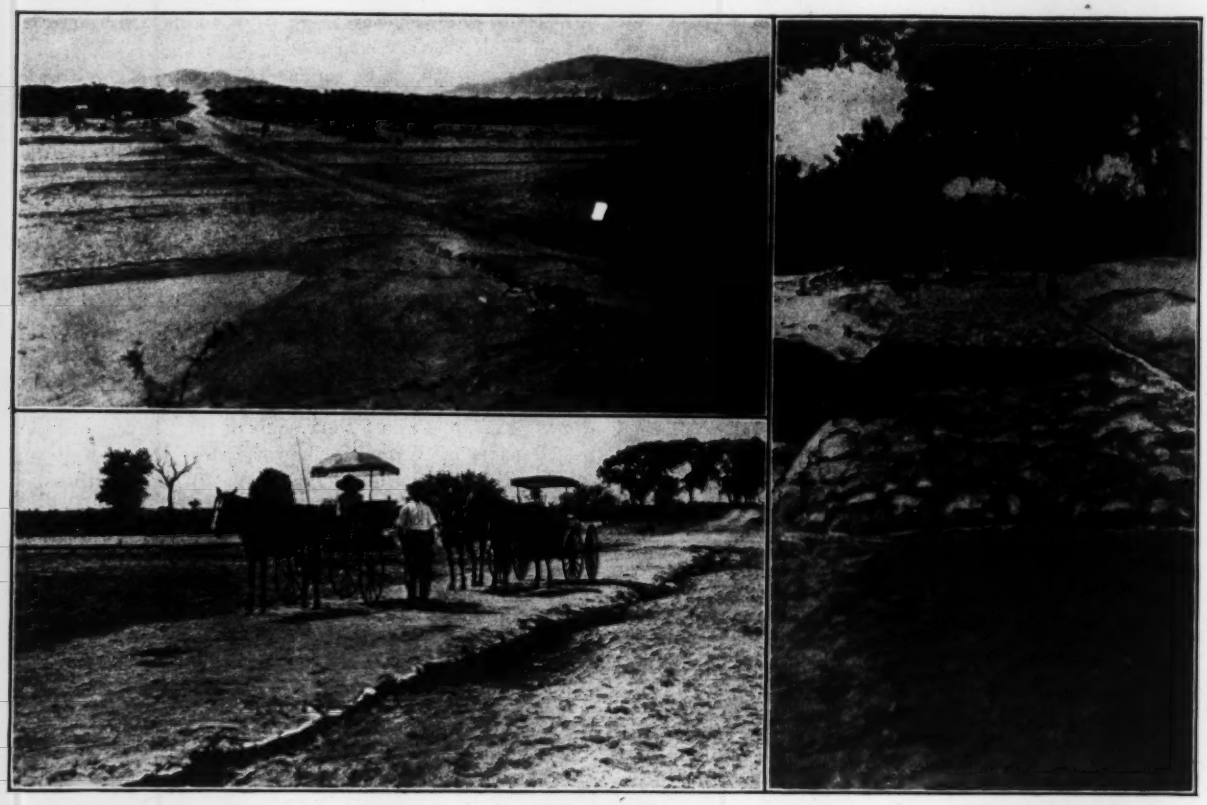

FIG. 6. VIEWS OF ARROYO CROSSING, PIMA COUNTY ROAD, ARIZONA, SHOWING METHOD OF CONSTRUCTION

ing up artificially for the deficient rainfall. These experiments will be watched with interest by highway engineers throughout the west.

Further Notes on Maintenance

The dirt roads of the Imperial Valley in California are maintained by thoroughly wetting them with water from nearby irrigation ditches and then dragging them as is done in the more humid parts of the country after rainstorms. The method is to divide the road along the center line by a dike and irrigate and drag one-half of the road at a time. Such a method, of course, is applicable only in level country and where plenty of water may be taken from irrigation ditches. It would hardly be economical where water had to be pumped or hauled any considerable distance.

In general, the scheme of maintenance of the dirt and gravel roads of the arid West, as already noted, is merely to supply new material as the old blows away in dust. This means the entire resurfacing every three, four or more years, according to the amount and kind of traffic. It is evident that the harder, coarser gravels give the best satisfaction. Between times of resurfacing the roads have to be occasionally recrowned and the ruts removed. This is generally done as frequently as can be afforded. Ordinarily, the roads become so badly rutted that a common road drag or light scraper is out of the question and large road planers or blade graders are used.

FIG. 7. WASHOUT ON A DESERT ROAD CAUSED BY FAILURE OF WOOD HEADWALL OF PIPE CULVERT

The cost of maintenance as also the cost of construction depends almost wholly on the cost of handling and hauling gravel. Passable material exists in almost every locality and since maintenance is chiefly the continual addition of new material economy of maintenance must be in handling and hauling methods. Motor trucks have been used very largely in Bexar County, Texas, and with economical results. On the other hand experience in Valencia County, New Mexico, with a motor truck has not been entirely satisfactory. When used in uninhabited country far from the base of supplies and from repair shops, or when the roads are soft or inches deep with dust, they have proved anything but satisfactory. In many parts of Arizona and New Mexico native, or rather Mexican, teams and teamsters may be employed for very low rates, at so much per cubic yard per mile hauled, so that motor trucks cannot compete on the basis of economy unless they are in service nearly all the time.

Notwithstanding that most of the roads described are in desert country a great deal of attention has to be paid to drainage. When rains do occur they are generally torrential in character and water courses normally dry

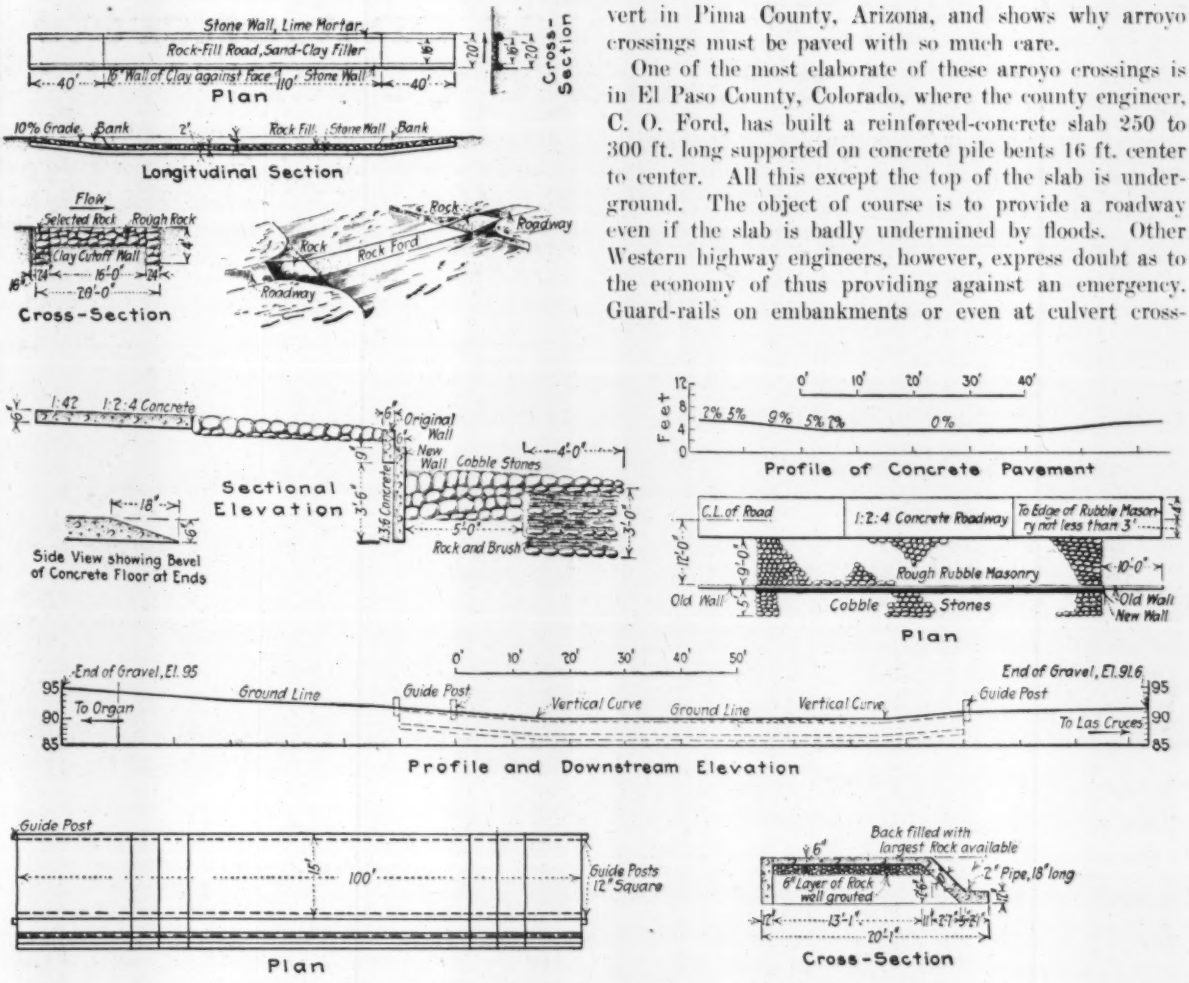

FIG. 8. DETAILS OF THREE ARROYO CROSSINGS ON DESERT ROADS

become small-sized rivers for short periods of time. Bridges and culverts of a permanent type are something of a luxury. In many sections the hauling costs on cement and steel are prohibitive. Culverts on much new construction are of the galvanized, corrugated-iron pipe type even in sizes up to six or more feet in diameter.

Highway structures peculiar to the arid West are paved sections of road across dry washes or arroyos, which are overflowed in times of flood. They take the places of very large culverts or bridges which would perhaps be in service but a few days each year. These overflow sections of road are known by various names: weirs, spillways, grade crossings (of arroyos) and fords. They are not really fords since they are dry the greater part of the year.

Arroyo or Dry Crossings

These arroyo crossings vary all the way from dished sections of road, unpaved and but a few feet across, to elaborately paved and protected sections of road over 100 ft. long, as shown in Fig. 8. It is very necessary to protect both the upstream and downstream sides of the overflow section from scour, for the adobe soil through which many of these roads are built erodes very rapidly in flowing water. Fig. 7 shows what happened to an improperly protected upstream end of a 48-in. pipe culvert in Pima County, Arizona, and shows why arroyo crossings must be paved with so much care.

One of the most elaborate of these arroyo crossings is in El Paso County, Colorado, where the county engineer, C. O. Ford, has built a reinforced-concrete slab 250 to 300 ft. long supported on concrete pile bents 16 ft. center to center. All this except the top of the slab is underground. The object of course is to provide a roadway even if the slab is badly undermined by floods. Other Western highway engineers, however, express doubt as to the economy of thus providing against an emergency. Guard-rails on embankments or even at culvert crossings are practically an unknown luxury throughout the greater part of the far West. The absence of guard-rails and frequent dips in the roadbed at arroyo crossings tend to bring about more cautious and less speedy driving of automobiles than is customary in the East. In fact one advantage claimed for the arroyo crossings described is that an automobile driver must slow up in passing one, and that thus enforcing moderate speed helps materially to prevent the destruction of the gravel road, which is extremely rapid under high-speed automobile traffic.

The American Fuel-Briquet Industry in 1915 produced 221,537 tons of briquets, valued at $1,035,700, according to a recent report by C. E. Lesher, of the United States Geological Survey. The figure was exceeded, however, in 1914. Most of the mechanical difficulties of manufacture have been solved in this country, and the future growth of the industry now depends upon the development of markets for the product. The producing plants are, however, so widely distributed and the total production is so small compared with that of other kinds of fuel that the conditions affecting the market for the output of each plant are more or less local and peculiar. There were 15 briquetting plants in operation in the United States in 1915, one less than in 1914. One new plant in California reported an output in 1915, and two plants—one in New Jersey, and one in New York—ceased operations. The greatest increase in output was made on the Pacific Coast, the Central States recording little change and the Eastern States a large decline in output.

Effect of Sewer Gas on Concrete in a Chicago Sewer

A new sewer now under construction by the Sanitary District of Chicago in the stockyards district is novel for that locality in that it has a brick arch on a tile-lined reinforced-concrete invert. In deciding upon this type of design the authorities were influenced partly by a condition of deterioration in the concrete of the arch of a neighboring sewer. The condition of the concrete sewer and the selection, after some controversy, of the brick-arch design for the new work have had considerable local publicity, and in order to spread the case before the profession in general, *Engineering News* presents the situation in two ways: First, a report on the condition of the existing concrete sewer, by Benjamin Brooks, of the staff of the International Clay Products Bureau, who made an investigation of that structure and its surroundings and testified as to that investigation before the trustees of the Sanitary District; second, a general description of the new sewer and a personal interpretation of the conditions in the existing sewer by a member of the editorial staff of this journal. Editorial comment appears on another page.

Sewer Gas Affects Concrete Above Water Line

By Benjamin Brooks*

Disintegration of concrete in a large sewer in Chicago offers points of interest both to chemical engineers and to sanitary engineers. The sewer is in Western Ave., is about 12 ft. in diameter, of the comparatively massive monolithic type, and was built about seven years ago.

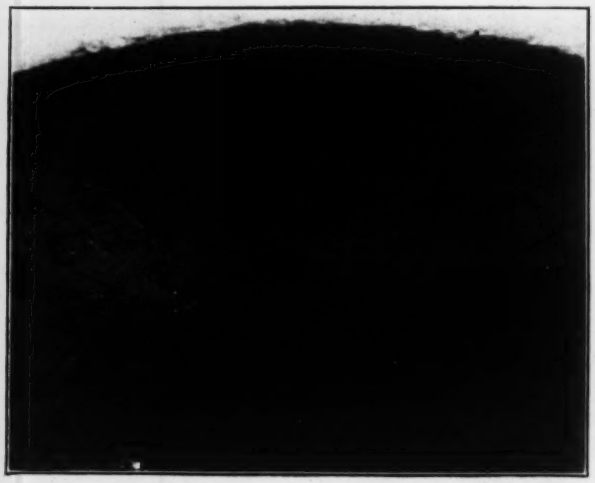

FIG. 1. CONCRETE ARCH OUTSIDE OF AFFECTED ZONE

The concrete is composed of portland cement, sand and gravel. The section where the destruction has been particularly noticed and studied is a stretch of about 500 ft. extending from the mouth of the sewer on the Drainage Canal.

*Engineer, International Clay Products Bureau, 204 New York Life Building, Kansas City, Mo.

The concrete immediately about the mouth of the sewer is hard and dense. At 15 ft. from the mouth the roof begins to show patches of whitish crumbly crust easily rubbed off by the hands, but the concrete beneath this crust is perfectly sound. Proceeding into the sewer, this incrustation rapidly assumes the character of a disintegration of the concrete, but is apparently only down to

FIG. 2. DISINTEGRATED SURFACE OF CONCRETE ARCH AND BOTTOM OF VENTILATING MANHOLE
The flat, loose patches here are probably scraps of the tar paper that lined the forms

the water line. First, it appears like fine, moist gray moss about ½-in. long, which under a pocket magnifying glass shows a distinctly crystalline structure. Farther on, this crystalline structure gives way to a surface of loose gravel held in a wet muddy matrix like half-set plaster of paris, pieces falling at the slightest touch. All these changes occur within about 100 ft. of the sewer mouth. At the same time the concrete in this same part of the sewer, but *below* the normal water line, is smooth and hard, as far as could be determined by pounding it with hammers and scraping it with bars.

Analyses by the Kansas City Testing Laboratory of the sound concrete and the disintegrated concrete from the tunnel roof are given in the accompanying table.

ANALYSES OF CONCRETE IN WESTERN AVE. SEWER AT CHICAGO

Sample	Sound Concrete, %	Decomposing Concrete, %
Moisture	2.13	4.12
Loss on ignition	19.81	20.21
Silica (SiO_2)	36.38	30.75
Iron (Fe_2O_3)	3.53	1.32
Alumina (Al_2O_3)	9.04	6.40
Lime (CaO)	23.36	17.30
Magnesia (MgO)	3.38	3.49
Sulphur (SO_3)	2.37	16.62
	100.00	100.21
Water soluble	5.12	28.96

Sulphur (SO_3)

In sound concrete	2.37%
In decomposing concrete	16.62%
In 200-mesh material from decomposing concrete	46.30%
In ¼-in. material from decomposing concrete	11.44%

The first noticeable feature of this comparison is that the concrete, from some source or other, has absorbed about six times its original content of SO_3. This original 2.37% of SO_3 is not abnormal when it is remembered that the "sound" concrete was not entirely free from the

whitish crust, a quantity of which was knocked down by heads and shoulders, working in the limited space available, and mixed with the sample. The moisture and organic matter have naturally increased. The percentage of other items has naturally decreased proportionately, owing to the large increase in SO_3. Some of the lime, being changed into soluble sulphate, has evidently washed away. Different theories as to the cause of this condition may be outlined as follows:

Since the city had some difficulty with the contractor who built this sewer, the first explanation of the disintegration of concrete was poor workmanship in the beginning. But no error of this kind would make *all* of the lower three-quarters of a concrete tube good and *all* of the upper one-quarter bad. The disintegration, once started at the point described, is very uniform and complete, not blotchy as in the case of uneven mixtures.

Another theory advanced is that the sewage is unusual in character on account of some acid factory tributary[1]

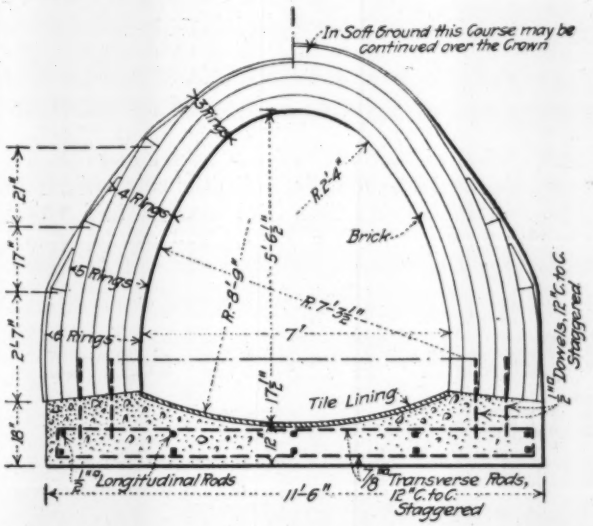

FIG. 3. SEMI-ELLIPTICAL BRICK SEWER ON CONCRETE BASE FOR STOCKYARDS INTERCEPTOR, CHICAGO

to it; but it must be evident that by the time a sewer becomes 10 or 12 ft. in diameter its proportionate length assures that it takes sewage from wide areas of various character. As a matter of fact, chemical analysis of the sewage shows it to be slightly alkaline—only 350 parts per million, expressed in terms of calcium carbonate.

A theory advanced by Mariner & Hoskins, chemists, of Chicago, is that some soluble substance in the soil surrounding the sewer leaches through it, concentrates by evaporation on the inner surface and destroys the concrete. Soil samples taken from excavations for the new ventilators were submitted to this firm for analysis and were found to contain about 0.1% sulphates of magnesium and calcium. This possibly is enough alkali to affect the concrete if concentrated by evaporation and would explain most features of the case; but it does not account for the circumstances that, in the part of the sewer lying under street pavements properly drained by the usual catchbasins and vitrified pipe to the sewer and where the opportunity for evaporation is small, the disintegration is much farther advanced than in that part of the sewer running under the power-plant lawns, where the rain would be more apt to seep through the concrete and where the fresh air has more access to the sewer for evaporation.

The solution of this disintegration of concrete can evidently be found by discovering the source and chemical progress of the excessive amount of sulphur. The most obvious and prolific source of sulphur in connection with a sewer is the hydrogen sulphide which is evolved naturally from sewage during decomposition. The sulphurous gas is oxidized by the air so as to form sulphuric acid, which immediately attacks the lime in the cement and converts it into soluble calcium sulphate. This sulphate, being of larger volume than the original lime, disrupts the concrete in forming, just as alkali crystals or repeated freezings sometimes do. Then it gradually dissolves away, leaving the aggregate to fall out.

A sample of the atmosphere taken just under the sewer arch and about 150 ft. from the mouth showed 2.45 mg. of sulphur per liter of air. This is equivalent to 0.17% by volume in the form of sulphur dioxide or hydrogen sulphide. One part in a million of hydrogen sulphide is enough to be plainly evident to the senses, and 0.17% is sufficient to be very active chemically. In the form of sulphur dioxide it might be even more corrosive.

[1]The Drainage District engineers state that they never have heard of any acid factory in the drainage area of the sewer nor of any theory ascribing the deterioration of the concrete to that cause.—Editor.

New and Old Sewers Taking Stockyard Sewage

[STAFF ARTICLE]

The stockyards intercepting sewer now being built by the Sanitary District of Chicago has a brick arch on a reinforced-concrete base forming the invert, although most of the numerous intercepting sewers built by the District are of concrete throughout (either plain or reinforced).

This new sewer extends along the south bank of the Bubbly Creek arm of the Chicago River from Ashland Ave. west to the Western Ave. sewer, a distance of about ½ mi. Its east end is a connection with the Ashland Ave. sewer, though an extension 1 mi. east to Halsted St. is projected, and it will also connect with the Robey St. sewer, besides carrying packing-house wastes from the stockyards district. The sewer will supersede the foul and practically stagnant open channel of Bubbly Creek, which is really a huge septic tank. This part of the creek is now being filled.

The sewer is of semi-elliptical section, as shown in the accompanying drawing, and is in two sizes: 9x9 ft. (for 770 ft.) and 7x7 ft. (for 2,675 ft.). These sections have four and three brick rings in the crown, an additional ring being added if necessary in soft ground. The base is a reinforced-concrete slab, the sides of which form the footings for the arch, while the middle portion of the top is concave to form the invert. This invert will be lined with 1-in. vitrified tile blocks 11¾x24 in. (laid in cement mortar) in order to form a smooth floor and to protect the concrete.

Alternative designs were prepared for all reinforced-concrete and combined brick and concrete construction. The sewage to be carried is exceptionally strong in organic matter, but is fairly fresh. To avoid any possibility of gases from this attacking and causing disintegration

of concrete, the engineers proposed a complete tile lining for the reinforced concrete, the tiles for the sides and arch to have dovetailed ribs keying them to the concrete. The lowest bids for these designs (all by the same bidder) were $91,377 for the reinforced-concrete design with tile lining on the invert, $100,649 for this design with complete tile lining, and $104,051 for the brick and concrete design with tile lining on the invert. The engineers suggested the use of concrete, with a complete tile lining, as an insurance worth the additional cost. The trustees of the Sanitary District, however, adopted the brick and concrete design, partly in view of the fact that brick is made and laid by Chicago manufacturers and union labor, while the difference in cost was small.

In the public hearings as to the desirability of using concrete or brick, stress was laid upon the condition of concrete in the Western Ave. sewer, which at present forms the west outlet from Bubbly Creek to the main channel of the south branch of the river. This carries sewage of a very different character from that which the new sewer will carry, the former being highly septic and being in fact the effluent from what might be called the largest septic tank in the world, receiving over half the entire waste from the packing-house district.

The period of detention in Bubbly Creek is probably from 36 to 48 hr., so that the sewage on reaching the existing conduit is full of hydrogen sulphide. The sewage is therefore most unusual in character and is unique in Chicago, if not in the United States. Inspection shows that action on the concrete has been entirely above the water line. Owing to hydraulic conditions, the flow through this sewer has been more variable than was anticipated, and consequently some deposits have occurred which in turn have produced hydrogen sulphide. The action appears to be largely on the roof, or under side of the arch.

The Western Ave. sewer is of elliptical section, 12 ft. wide and 16 ft. high, with concrete 12 in. thick in the sides and crown. There is no reinforcement, except for about 660 ft., where it passes under private property. This sewer was built in 1909. On 900-ft. of the outfall the surface of the concrete is now largely covered with efflorescence, and there are occasional hollow pockets in the concrete as well as patches where the aggregate is exposed. Some of these places are due apparently to poor workmanship, the aggregate having been left against the form without the proper amount of mortar.

In other patches, however, the concrete is soft and disintegrated, showing high sulphates. This condition is claimed to be due to the action of the sulphuric acid in the gases, causing an efflorescence that expands and gradually scales off the surface, exposing the fresh material to attack. Other parts of the surface still show the original form marks and cement skin, and patches of the tar paper that covered the forms. The disintegration is most marked where ventilation was lacking and does not occur near the inlet and outlet. As originally built, there were no manholes, but these have been provided within the past three months in order to afford ventilation and relieve the foul conditions at the middle of the sewer.

Combination of Plane Table and Transit for Topography

By Holbert T. Johnson*

SYNOPSIS—Use in combination of plane table for plotting in the field and a transit for traversing and stadia work. Analysis to show labor saving in combination.

For ordinary topographic surveys either the transit or the plane table is the instrument usually selected. There has been a good deal of discussion as to which is the best instrument, and it is not the purpose of this article to add to that discussion; for it is apparent to anyone who has used both instruments that each has its share of faults and virtues, and that in general the advantage which one has the other lacks. In an endeavor to combine the good points and eliminate the faults the writer has developed a new method in which both instruments are used in combination. It is not applicable to all topographic work, but under the conditions usually met with, it is believed that it will be found faster, cheaper and more accurate than any of the ordinary methods.

Instruments and Party Required

The instruments used for the combination method are a light transit with a sensitive telescope bubble and a full vertical circle with vernier reading to minutes, a 24x30-in. plane table and a plane-table alidade with striding level and a compass needle.

A flat scale used for plotting distances on the plane-table sheet is notched slightly at the zero mark and a small rubber loop, just long enough to stretch to the notch, is attached to the scale about ¼ in. from its edge. When a pin is stuck through the loop into the board, the scale can be revolved in any direction and the zero point will be held at the pin. The scale is also provided with two sights parallel to its graduated edge. Two pins can be used for these but folding sights are preferable. (See Fig. 2.)

A recommended type of stadia rod is a ¾-in. soft pine board, 14 ft. long, 3½ in. wide at the bottom and 2½ in. at the top. The divisions are as shown in the accompanying sketch, Fig. 1. As there are no figures on the rod it takes some little time for a beginner to read it rapidly, but after getting accustomed to it I have never seen an instrumentman who would use any other. When the heat waves are bad, this rod can be read at distances where the ordinary stadia rod markings would be indistinguishable.

The party for the combination method will usually consist of three rodmen, a transitman and the topographer. If the shots are taken oftener than every 200 ft. two rodmen will be sufficient. If the land requires many vertical angle shots a recorder should be added to the party.

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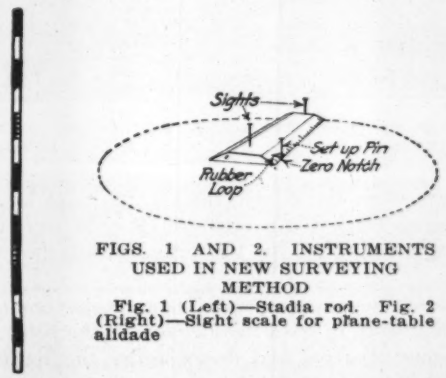
TABLE SHOWING OPERATIONS OF TWO MEN AT THE INSTRUMENT IN TAKING A SIDE-SHOT

	Transit Method		Plane-Table Method		Combination Method	
	Transitman (telescope level)	Recorder	Topographer	Recorder	Transitman (telescope level)	Topographer
Ordinary shot in flat country. Rod cut by upper, lower or middle wire	1. Sights on rod 2. Reads rod aloud 3. Turns tangent screw till distance can be read 4. Reads distance aloud 5. Moves tangent screw till wire cuts rod at original reading (this leaves telescope level for next shot) 6. Signals rodman "all right" 7. Reads azimuth aloud 8. Calls special features	1. Records rod 2. Records distance 3. Computes elevation 4. Records azimuth 5. Records features and makes sketches	1. Sights on rod, keeping ruler edge of alidade against setup needle 2. Levels up telescope bubble 3. Reads rod aloud 4. Moves tangent screw till distance can be read 5. Reads distance aloud 6. Signals rodman "all right" 7. Draws line along alidade edge 8. Moves alidade 9. Sets zero of scale at setup needle and plots distance 10. Records elevation on the map 11. Draws in contours and special features	1. Records rod 2. Records distance 3. Computes elevation 4. Calls elevation	1. Sights on rod 2. Reads rod 3. Turns tangent screw until distance can be read 4. Reads distance aloud 5. Turns tangent screw until wire cuts rod at former rod reading (leaving telescope level) 6. Records distance 7. Records rod reading 8. Figures elevation calling it aloud	1. Sights on rod with scale 2. Plots distance 3. Signals rodman "all right" 4. Records elevation on map 5. Draws in contours and special features
Vertical Angle shot	1. Sights on rod 2. Turns tangent screw till distance can be read 3. Reads distance aloud 4. Sights middle wire on rod reading used for obtaining the vertical angle 5. Signals rodman "all right" 6. Reads azimuth aloud 7. Reads vertical angle aloud 8. Calls special features	1. Records distance 2. Records azimuth 3. Records vertical angle 4. Records special features and makes sketches 5. Computes difference of elevation 6. Computes elevation	1. Sights on rod, keeping ruler edge of alidade against setup needle 2. Turns tangent screw until distance can be read 3. Reads distance 4. Sights middle wire on rod reading used for obtaining the vertical angle 5. Signals rodman "all right" 6. Reads angle A on vertical circle of alidade aloud 7. Levels up telescope 8. Reads angle B on vertical circle of alidade 9. Draws line along alidade edge 10. Moves alidade 11. Sets zero of scale at needle and plots distance 12. Records elevation on the map 13. Draws in contours and special features	1. Records distance 2. Records angle A 3. Records angle B 4. Computes vertical angle difference of A and B 5. Computes difference of elevation 6. Records difference of elevation 7. Computes and calls elevation	1. Sights on rod 2. Turns tangent screw until distance can be read 3. Reads distance aloud 4. Sights middle wire on rod reading used for vertical angle 5. Records distance 6. Reads vertical angle aloud 7. Records vertical angle 8. Records difference of elevation 9. Figures elevation calling it	1. Sights on rod with scale 2. Plots distance 3. Signals rodman "all right" 4. Figures difference of elevation calling it aloud 5. Records elevation on map 6. Draws in contours and special features

The field procedure is as follows: Set up, level and orient the plane table, using the alidade. The elevation of the plane table H.I. is then obtained by a reading on a turn or benchmark, and the alidade removed from the board. A pin is stuck at the point on the map representing the station occupied and the sight scale pivoted to it. The transit is set up about 5 ft. from the plane table and its H.I. obtained by another reading on the turn or benchmark. This elevation is checked with the plane table H.I. by measuring the difference in the elevation of the instruments.

When a rodman holds up his rod the transitman takes the rod reading and notes the distance, calling the distance aloud; the topographer sights on the rod with the scale, signals the rodman "all right" and plots the point at the proper distance on the graduated edge of the scale. By this time the transitman has computed the elevation, and he calls it out, whereupon the topographer marks it on the plotted point. A good plan is to record the even feet to the left and the tenths to the right of the point so that besides showing the location of the shot it will also serve as a decimal point for the figures.



FIGS. 1 AND 2. INSTRUMENTS USED IN NEW SURVEYING METHOD

Fig. 1 (Left)—Stadia rod. Fig. 2 (Right)—Sight scale for plane-table alidade

The transitman records the rod readings, distances and elevations, and the topographer sketches in the contours and special features as fast as the shots are taken. As the directions are all obtained with the sight scale no horizontal angles are read on the transit.

The turns are taken as in plane-table work, the alidade again being used to level and orient the board and to check the turn elevations and distances obtained with the transit. This checking saves much time and labor as many errors are picked up which would otherwise be carried to the side-shots following. Checks for elevation and location should be made about every mile. In this distance the error in levels should not exceed 0.2 ft. and the error of closure of the plane-table traverse about 1 in 300. The fact that the transit is not coincident with the plane table introduces a slight error, but where scales of 1 in. = 200 ft. or more are used this is negligible.

Comparison of Transit and Plane-Table Methods

To show the distribution of labor and the comparative field speeds of the three methods the accompanying table analyzing the operations necessary for each side-shot has been arranged. It will be seen from this that the transit and the combination methods are just about equal in field speed, and that both are far ahead of the plane-table method.

In the office the transit topographer does not have such plain sailing. In rough country, unless the shots have been well chosen and close together, it is often necessary to send a man again into the field to sketch in contours at doubtful places.

The office work for the plane-table and the combination methods consists merely in checking and inking the elevations and tracing the topography. In steep country the topographer does not always have time to draw in all of the contours, but there will be enough so that the rest can be accurately located. The field sheets made by the combination method are much cleaner than the regular plane-table sheets, as the substitution of the sight scale does away with the brassy smear from the bottom of the alidade.

The elevations obtained with the transit are usually more accurate than those taken with the alidade, because its bubbles are more sensitive. The use of both instruments combines this advantage with that of field sketching. In this regard it is even better than the plane table, as the topographer has more time to watch the country and to do his sketching.

With the plane-table or combination method it is often possible to pick up errors on side-shots which would pass unnoticed in transit work. Errors on turns which are often a source of trouble in plane-table work are practically eliminated by the use of the two instruments.

In windy weather the combination method is much superior to the plane table, as the transit is less sensitive to the wind. I have seen a plane table vibrate so in a wind that the alidade would "walk" clear across it, while a transit alongside was hardly affected at all.

Topographic Survey of Duck Valley Project

During the season of 1915, while investigating an irrigation project for the Duck Valley Indian Reservation in northern Nevada, the writer made a survey of a large tract of the irrigable lands by the method described above.

A 2-ft. contour interval and a scale of 500 ft. = 1 in. were decided upon for the topography, which was to be used for designing the entire canal and lateral system. The United States Land Office had made an accurate resurvey of all the land on the reservation in 1911 and 1912, and this was used as a basis for all topographic work.

A system of benchmark levels for the topographic control was run in from a United States Geological Survey benchmark on one side of the tract. A 25-mi. primary circuit was run and secondary circuits closed on this until elevations had been established at all of the quarter-section and section corners on the north and south section lines. There were in all about 105 mi. of these levels run. All the circuits closed well within the limits of the formula: Allowable error in feet = $0.05\sqrt{}$ dist. in mi. The best day's record for this work was 11¼ mi. in 7½ hr. The last 6 mi. was in very level country and was run in just three hours. The total distance walked on this day was 17 mi. The party consisted of a level man, two rodmen and a man to hunt corners.

Fieldwork on the topographic survey was started about the middle of April and finished early in August. The survey covered in all 47,500 acres, most of which is in rolling country. The land was covered with sagebrush about 3 ft. high, which interfered very little with the work. A large part of the area was over two hours' drive from the nearest camping place, so that considerable time was lost in traveling.

The average day's run was about 650 acres with 500 side-shots. On one day when the air was exceptionally clear and the country very level, we used three men on horseback for rodmen and took 500 side-shots and 2,200 acres of topography in seven hours' working time. Most of the country was so rough, however, that the shots could not be taken more than 250 ft. apart.

The salaries paid on this work were as follows:

	Per Month		Per Month
Topographer	$150	Rodman	$75
Transitman	90	Teamster	65
Head rodman	90	Cook	75

Subsistence amounted to 50c. a day and was paid for by each man.

The total cost of the survey including supplies, camp expense, control levels, fieldwork and the completed tracings was $2,460, and the total number of side-shots taken was 33,000. This gives a unit cost of 5.2c. per acre and an average of 0.7 shots per acre.

The cost of topography of similar accuracy, taken with the plane-table or the transit method usually amounts to at least 8c. an acre, so that the combination method has a decided advantage in this respect over either of them.

Rail Joints Subjected to a Severe Test

The accompanying view shows one of the extensive washouts that occurred along the line of the Southern Ry. at the time of the heavy floods near Asheville, N. C., in July. As shown in the photograph, when the earth embankment was washed away the track was left suspended, was moved downstream some distance from its original position and was left hanging from the embankments on

WASHOUT ON THE SOUTHERN RY., 21 MI. EAST OF ASHEVILLE, N. C., ON THE BLUE RIDGE DIVIDE

either side when the floods subsided, the total length of the hanging track being over 250 ft. As seen in the photograph, nearly all the ties remained attached to the rails.

This long "suspension bridge" is a graphic testimonial of excellent work in track maintenance at the point where the washout occurred and demonstrates the remarkable strength of modern rail joints and track bolts. We are indebted for the photograph to the courtesy of F. A. Weakley, of the United States Engineer Office at Nashville, Tenn.

Granite-Block Pavement for a Residential Street

An innovation in the paving practice of the Borough of Brooklyn, New York City, has been introduced in the use of a 4-in. grouted granite-block pavement on a section of Bedford Ave., which is here a strictly residential street of good-class apartment houses, although a much used through route. This is the first pavement of 4-in. granite block to be laid in Greater New York other than the few hundred yards of experimental pavement on Second Ave., Manhattan, and the specifications contain a number of interesting features.

The specified dimensions of the blocks are: Not less than 7 nor more than 11 in. long, not less than 3¾ nor more than 4¼ in. wide, and not less than 3¾ nor more than 4¼ in. deep. They are so dressed as to be laid with ⅜-in. joints. The concrete base is of 1:3:5 mix, 6 in. thick, and on this is spread a ¾-in. bed of clean, coarse, dry sand mixed with portland cement in proportions of one part cement to four of sand. After the blocks are laid and rammed, they are sprinkled with clean water and the joints then filled with 1:1 grout, which is broomed into the joints. The operation is continued until the joints will take no more grout and are filled flush with the tops of the blocks.

FIG. 1. MIXING DRY SAND AND CEMENT FOR ¾-IN. BEDDING COURSE

FIG. 2. LAYING 4-IN. GRANITE-BLOCK PAVEMENT ON BEDFORD AVE., BROOKLYN

Another significant feature of the specifications is that which requires the contractor to complete the pavement on one-half of the street and allow time for this completed half to be opened to traffic before beginning work on the other half. This requirement is necessary, as Bedford Ave. is a connecting link between important parts of the borough and there is no parallel street to which to divert traffic. The monolithic character of the pavement is not lessened by this method of construction, because of the care taken to dovetail the two halves of the pavement together and because the thin mortar bed (¾ in.) prevents any appreciable difference in the setting of the blocks on either side of the center line. Some of the more important contract specifications are given in full:

The granite paving blocks shall be of fine-grained granite showing an even distribution of constituent minerals, of uniform quality and texture, without seams, scales or disintegration, free from an excess of mica or feldspar. The granite shall have a toughness of not less than 9 and a French coefficient of wear of not less than 11, as determined by the method employed by the United States Office of Public Roads and Rural Engineering.

The blocks shall be laid not more than 25 ft. in advance of the ramming; no blocks or other material must be thrown or deposited in such manner as to displace the blocks on the face of the work, and no carting will be permitted over the surface of any unfinished portion of the pavement.

The grout shall be mixed in a machine mixer, approved by the engineer. The character of the mixer shall be such that the mixing of the grout shall be continued until the instant that it is poured into the joints.

The filling of the joints shall be carried forward until an advance area of 15 or 20 yd. has been grouted, when the same force and appliances shall be used to regrout the same space in a like manner.

The work shall be kept lightly sprinkled with water ahead of the sweepers, to avoid the possibility of the grout becoming too thick at any point.

To insure penetration of the grout into the joints of the pavement a squeegee 15 or 18 in. in length, in addition to the brooms, shall be used upon the last application of the grout.

After the grout between the joints has fully subsided and the initial set is taking place, the whole surface shall be lightly sprinkled with water and the surplus grout left on top shall be swept off.

After the grouting is completed and a sufficient time for hardening has elapsed, so that a coating of sand will not absorb moisture from the cement mixture, ½ in. of sand shall be spread over the whole surface.

Should the bond between the blocks become broken before the work is accepted, the joints shall be cleaned out, even if it is necessary to take up and relay the blocks, and such defective work shall be regrouted and relaid.

Before letting this contract the chief engineer of the Bureau of Highways of the Borough of Brooklyn, H. H. Schmidt, called the contractors before him and distinctly stated that specifications would be strictly enforced to the letter, for the pavement is in the nature of an experimental one and its adaptation to conditions will be closely watched. The contract price was $4.30 per sq.yd.

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The Foreign Trade of the United States for the year ending June 30, 1916, exceeded 6½ billion dollars. This is far in excess of the foreign trade ever recorded by any other nation. It was not until the year 1913 that the total foreign trade of the United States exceeded four billion dollars. The most noteworthy feature of the year's trade, of course, is the enormous excess of exports over imports. The total exports from the United States were $4,345,000,000 and the imports were $2,180,000,000. Thus in a single year there was a balance of trade in favor of the United States of $2,165,000,000. Under normal conditions, international trade balances are settled by the transfer of gold; but if European countries, exports to whom produced this enormous debt to the United States, were to transmit gold to the United States in payment of this balance, their entire stock of gold would hardly cover a couple of years' business. The entire amount of gold held by the great government banks of Europe was estimated not long ago to be less than $4,000,000,000. It is out of the question, therefore, for Europe to pay its trade balance to the United States in gold for any considerable length of time. Europe's debt to the United States for this huge volume of exports has been settled by the transfer of a small quantity of gold, by the floating in the United States of foreign loans and by the issue of notes against American securities held abroad.

Timber and Gravel Coffer-Dams on the Troy Dam

By D. A. Watt*

SYNOPSIS—The dam across the Hudson River at Troy, N. Y., required a number of coffer-dams in order to lay dry successive sections of the river and to permit free flow and sectional construction. This article describes the timber box coffer-dams and the gravel fills built in front of the old dam just prior to removing it. A later article will be devoted to the sheetpile cylinder coffer-dams built around the lock site and around a part of the main dam.

The lock and dam at Troy, N. Y., built in 1913-15 by the United States with hired labor forces, forms the connection between the New York State Barge Canal and tidewater in the Hudson River. The new works were built to replace an existing timber dam and a lock of small dimensions, situated about a quarter of a mile downstream, and the channel improvement required as the last feature the removal of this dam, which was about 1,100 ft. in length. The construction thus had to be carried on in the pool water of the dam, which could not be dispensed with until the last moment, owing to the large and steady traffic passing into and out of the connecting state canals. All masonry is of concrete, founded on Hudson River shale. Fig. 1 shows the general plan of the structures and the location of the coffer-dams required to lay dry the construction areas. The lock, with a width of 45 ft., a length of main walls of about 624 ft. and a depth of 16 ft., on miter sills is on the Troy side of the river and is tied into the concrete gravity dam, varying from 17 to 50 ft. high, which extends across the river on the broken line shown. Its total length is about 1,291 ft.

The considerable depths of water in which the coffer-dams had to be constructed, varying from 10 to 27 ft. at low water, the exposure to floods and heavy drift ice and the limited time in which the whole work had to be completed enforced the adoption of some special expedients in design and construction. In all, eight different coffer-dams were employed, two being of timber framing, three of steel piling and the remainder banks of gravel and sand from the river bed. The natural conditions of low banks and flood ranges prescribed that these structures need not rise above the pool level more than 8 or 9 ft.; that where they were to remain over winter in any exposed position they must be of a type to withstand the attacks of floods and the battering of running ice; and that they must not occupy much of the river for any length of time, lest they aggravate flood conditions in the thickly settled neighborhood above.

Fig. 1 shows the numbers corresponding with the sequence in which the various coffer-dams were completed. No. 1, around the west end of the dam, and No. 3, inclosing the west section of the dam, are of the wood-frame box type; Nos. 6, 7 and 8, in front of the old dam and used to lay dry that structure prior to its destruction, are of gravel and sand fill. These five coffers will be described in this article. The remaining three dams, Nos.

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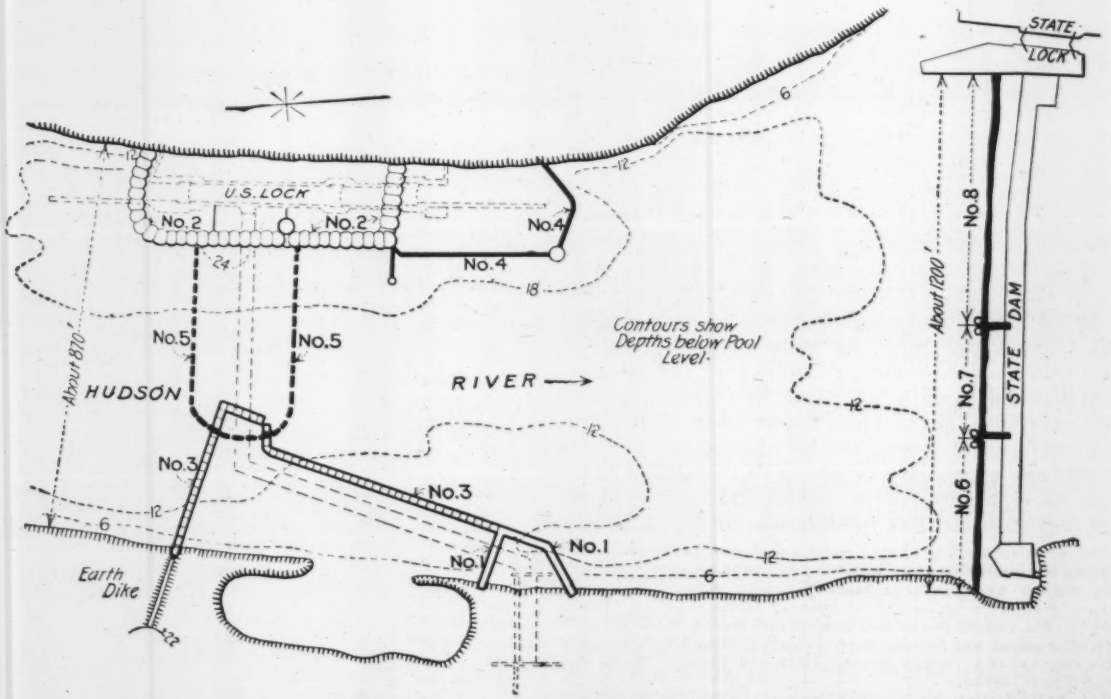


FIG. 1. GENERAL LOCATION OF COFFER-DAMS IN CONSTRUCTION OF LOCK AND DAMS AT TROY, N. Y.

2, 4 and 5, around the lock and the east part of the dam, are of sheetpiling of varying design, and their description will be reserved for a later article.

Main Timber Coffer-Dams

The first of the timber coffers is that shown in Fig. 1 as No. 1, inclosing the site for the bulkhead on the west bank, the general design being modeled on one that had been successfully used elsewhere. It was a timber struc-

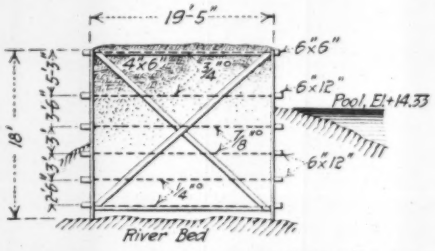


FIG. 2. SECTION THROUGH BULKHEAD COFFER-DAM (NO. 1)

ture of the box or wale-and-sheeting type, about 400 ft. long on the center line, and of cross-section as shown in Fig. 2.

The other wooden coffer, No. 3, inclosed the west arm of the dam. The first lock coffer-dam was at that time blocking up more than a third of the river's cross-section, and it was therefore desirable to block up as little more as possible during the season of high water. It was decided to begin the new coffer-dam as soon as the spring floods had gone, making it of a type that could be quickly constructed, and then to build the masonry in the inclosure as fast as possible and remove the coffer-dam before the fall and winter floods set in. The box type, as used for the bulkhead, was chosen as being simple and inexpensive, and the design was modified to suit the depth of water, which ran from about 11 to 17 ft. at low water (Fig. 3). The distance from the masonry was made such that concrete could be delivered by the floating mixers to towers and be spouted into the forms, requiring a rehandling only for the tops of the sections.

Building Timber Coffer-Dam from Barge

As the length of the structure was to be about 1,040 ft. some plan of construction had to be devised more speedy than the usual one of setting the walings one by one by a derrick boat. The method adopted consisted in laying horizontally and loosely the two separate sets of walings for a 20½-ft. section on two corresponding frames or cradles lying opposite each other on the deck of the working barge. Each cradle was provided with projecting arms bolted on at distances corresponding to the spacing of the walings and supporting the latter when the cradles were set up. The feet of the cradles were lashed to the deck of the barge by slack loops of wire line, and when all was ready the first cradle was hoisted to the vertical by the outer end and guyed in position. The opposite cradle was next hoisted; tie-rods and spreaders were attached, and connection was made to the preceding section; the new-made crib was then lifted slightly, allowing the cradles to clear and be lowered again to the deck of the barge. The fleet was then warped along until the new crib had slid into proper position over the side of the barge, and the operation of attaching another section was begun.

The maximum lift for the derrick boat in this movement was about 5 tons. The process is illustrated by Figs. 4, 5 and 6. The walings, spreaders, etc., were all cut to length and bored on shore, so that when stacked on a barge, the derrick boat had merely to lift the pieces one by one and set them in proper position on the cradles. The method was found to give excellent results. After the force became accustomed to the work, they could build and launch 100 ft. of straight cribwork in two 8-hr. shifts, using a single derrick boat. This rate, the maximum attained, would have been exceeded considerably but for the fact that the holes for the tie-rods, bored from ¼ to ½ in. large, according to the size of the rod, with the view of avoiding binding, were not always bored straight, although boring machines were used. There was therefore a good deal of delay in working the tie-rods through, especially at the joints. It is believed that the holes for such work should be made from ¾ to 1 in. larger in diameter than the rod, so as to prevent this difficulty.

For launching the cribwork two 12x12-in. timbers were hinged to the side of the barge, as shown in Fig. 6. Their ends rested on the bottom and slid over it as the barge moved along the line, the framework sliding in turn down the timbers and moving off their ends to rest upon the river bed. A special design of waling joint had to be made to avoid binding, as shown in Fig. 3. From ¾ to 1½-in. play had to be allowed for the free hinge movement of the walings during launching; and where the piece was halved on the outer side, one or two bolts, according to location, were used to prevent longitudinal splitting from the bending strain.

For the sizes of the woodwork it was assumed that hemlock or spruce would be used, this being the cheapest local timber. The length of sections was made 20½ ft., so as to use standard 22-ft. lengths, and the walings were spaced to secure as far as practicable an equal unit stress in each and in the sheathing. They were figured as simple beams of lengths equal to the distance between ad-

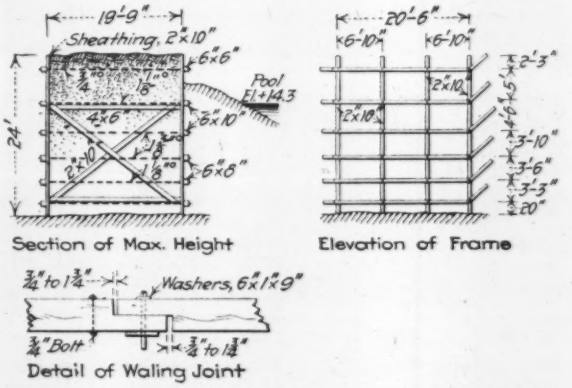
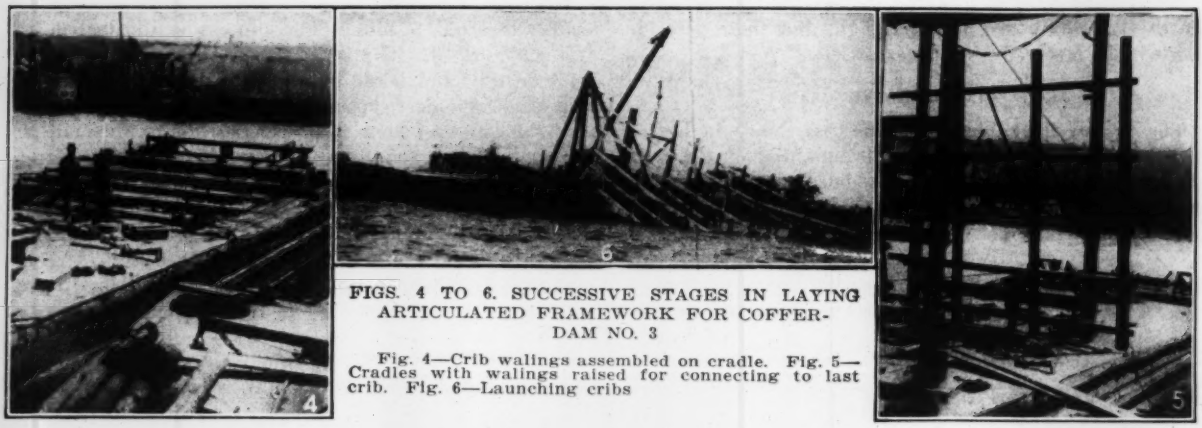


FIG. 3. DETAILS OF COFFER-DAM FOR WEST ARM OF DAM (NO. 3)

jacent tie-rods. The sheeting, which could more safely stand an overstrain without risk of disaster, was figured as continuous beams supported at each waling. The water elevation on the river side was taken as 14.33 (equivalent to pool level) and on the inside as below the base of the coffer. The extra loads induced by floods were to be covered by the general factor of safety. No reduction of loads on the timberwork from the support of the outside banking was allowed for, since the latter might



FIGS. 4 TO 6. SUCCESSIVE STAGES IN LAYING ARTICULATED FRAMEWORK FOR COFFER-DAM NO. 3

Fig. 4—Crib walings assembled on cradle. Fig. 5—Cradles with walings raised for connecting to last crib. Fig. 6—Launching cribs

be eroded more or less by floods, but the riverward pressure of the inside filling upon the walings was reduced by the opposing water pressure, making a considerable saving in sizes (see Fig. 3). The design reduced the total waling cross-section 25% as compared with that used for the same height on coffer No. 1. The weight of the filling (dredge-placed gravel and sand) was taken as 120 lb. per cu.ft. for the whole height, and its wedge of pressure against the sides as sliding on a base of inclination of 1 vertical to 2½ horizontal, the formula used being Rankine's. With the spacings shown, the stresses under the foregoing assumptions varied in the walings from 860 to 1,240 lb. per sq.in. and in the sheathing from 700 to 1,270 lb. per sq.in. The maximum stress allowed in the tie-rods was 21,000 lb. per sq.in. of the area at the root of the thread.

This coffer-dam fulfilled its purpose entirely satisfactorily. The timberwork was begun at the north-bank end on May 12, 1914, and was practically completed, including the sheeting, by July 1, using only one derrick boat. The sheeting accompanied the main timberwork, and the filling and banking by the hydraulic dredging followed, commencing on May 25 and being completed on July 31, ten days after unwatering had begun. The outer sheeting was driven only a foot or so into the river bed, as the latter was of gravel and the plank would not penetrate it far without splitting; the inner sheeting rested on the river bed. Pumping was begun on July 21 with one 10-in. and one 12-in. centrifugal pump, and the water was lowered sufficiently by July 29 to begin excavation on that date. The only leaks of consequence were through porous strata under the coffer at the outer angles. As the banking became seasoned there, this leakage gradually lessened until the 10-in. pump alone was able to keep the water down, although the lift between water surfaces was about 23 ft. and the area of the inclosure below the river level was between 6 and 7 acres. The masonry, about 800 ft. in length, was begun on Aug. 20 and completed on Oct. 24. No rise exceeding 2 ft. occurred during the life of the coffer. It was finally flooded through the north sluice on Oct. 26, and the greater portion was removed by the dredge and derrick boats before the following December.

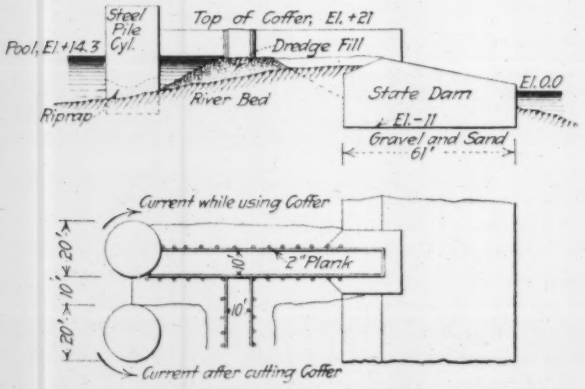


FIG. 7. DETAILS OF COFFER-DAMS IN FRONT OF OLD STATE DAM

The cost and materials were as follows:

Lumber (spruce), 170,000 ft. b.m. @ $27 f.o.b. Troy (about $4.60 per ft. run)	$4,590
Tie-rods, bolts, etc., 96,400 lb. @ about 3.85c. threaded, etc. (about $3.60 per ft. run)	3,700
Handling framing and placing lumber and placing iron (about $6 per ft. run, or $36.50 per M ft. b.m.)	6,200
Filling and banking by hydraulic dredge, 46,500 cu.yd. @ 12.2c. (about $5.60 per ft. run)	5,670
	$20,160
Length of coffer-dam, 1,040 ft.; cost per ft. run	$19.40

The foregoing are field costs of construction only and do not include depreciation charges nor supervision above the grades of foreman, nor district-office costs. Ordinary running repairs are included. The removal is not included, as it was done at odd times, and much of the filling and of the remains of the timberwork was washed away by floods.

Gravel Coffer-Dams at Old Dam

Subsequent to the construction of the dam inside the timber coffers described, the lock and end of the dam were built inside of the steel coffer-dams to be described in a later article. The last piece of work was the construction of the gravel dams just upstream of the old lock and dam.

These three coffer-dams (Nos. 6, 7 and 8) were used for the removal of the State dam and consisted of banks of river material, the bases being placed by the hydraulic dredge, and the tops in part by the dredge and in part by derrick boats. A cross-section of No. 6 is shown in Fig. 7, and the others were similar except that a plain gravel top was used, without plank. The removal of the old dam required the excavation of about 50,000 cu.yd. of riprap and timbers. All this work, including the making of the coffer-dams, had to be done during the winter of 1915-16, so as not to interrupt navigation.

The steel-pile cylinders were used to protect the corners from erosion before and after the cutting of the coffer-

dams. They were filled by the hydraulic dredge, after a 5-ft. layer of riprap had been thrown inside. No. 6 was closed first; and when the section of dam under its lee had been nearly all removed, No. 7 was completed. High water constantly troubled the work, but the banks stood safely although nearly topped more than once. Finally, on Dec. 26, 1915, about three weeks after the close of navigation, No. 6 had to be cut in order to save the rest of the work from a high flood. The head between the two pools was 11½ ft. at that time, and the 230 ft. of bank washed out in less than an hour, leaving a clear gap for the passage of the river and reducing the head to 5 or 5½ ft. The end of No. 7 was eroded somewhat by backlash, but further damage was stopped by riprap. This latter coffer was cut about a month later, and the river washed it away piecemeal.

No. 8 covered the remainder of the dam. After the usual vicissitudes from cold weather and high water, excavation was practically completed by the end of March. One of the highest floods in several years came a day or two later and, topping the bank, washed down its crest 10 to 15 ft. The difference of level between the water surfaces before the flood reached this crest was less than a foot, as compared with 11½ ft. when the work of removal had been begun about four months before.

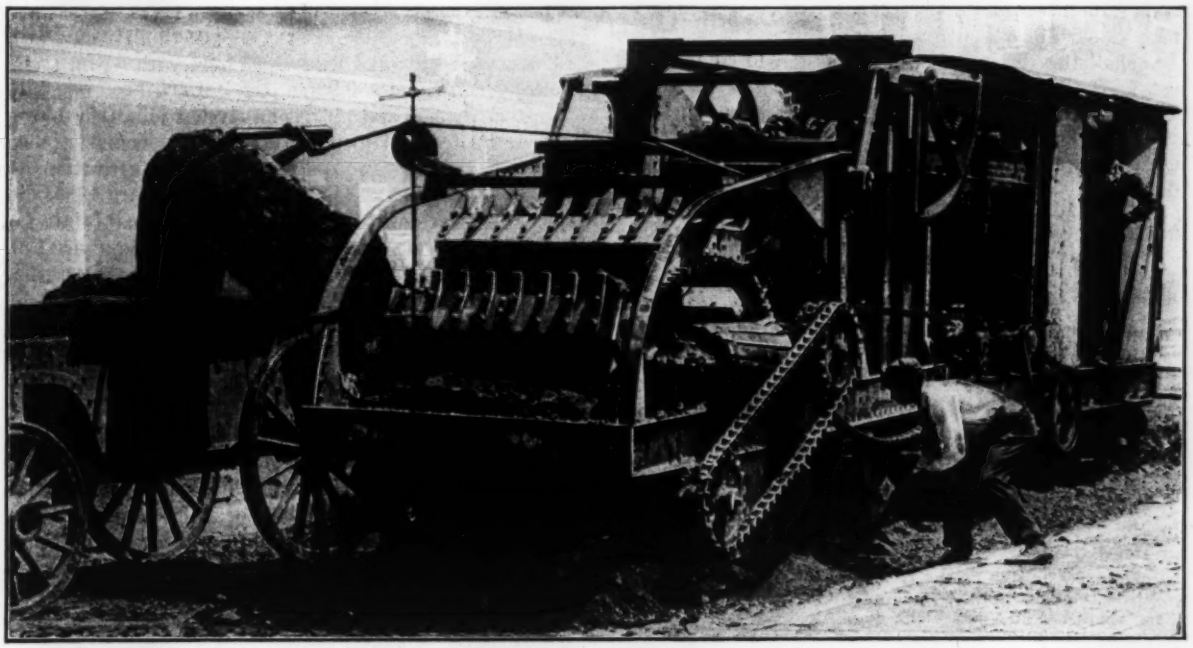
Engineers in Charge of Work

The work was performed under the supervision of General (then Colonel) W. M. Black, now Chief of Engineers, United States Army, with Captain R. D. Black as local engineer officer, succeeded in 1914 by Major M. J. McDonough. The fieldwork was commenced by Assistant Engineer A. C. Harper and continued by Junior Engineer J. J. McCabe. The mechanical work was under Wm. O. Quinlivan and the office work under Junior Engineer F. P. Fifer. The writer supervised the work locally.

Excavator for Street Grading

A new type of excavator and wagon-loader has been used in grading North Broadway, Chicago, south from Devon Ave. The machine has a steel frame mounted on caterpillars and having a wide-tired steering wheel at the rear. It is shown in the accompanying view.

At the forward end is the excavating device. This consists of a pair of circular frames carrying a series of horizontal shallow buckets fitted with teeth on the cutting edges. The buckets passing over the top of the circular frames drop their contents upon a belt conveyor. At the side of the machine this conveyor is carried by an inclined boom, high enough for wagons to drive beneath it and receive the material. The front portion of the frame, carrying the excavator wheel, is pivoted and is adjustable vertically to regulate the depth of cut and also to enable the buckets to be raised so as to clear manholes and street intersections. This frame is raised and lowered by means of the pinion and quadrant gear shown.



EXCAVATING MACHINE FOR GRADING STREETS

On the rear end of the main frame is an opposed-cylinder gasoline engine which operates the excavating and propelling mechanism through gears and chain drives. The machine could grade a length of about two blocks per day, and kept 35 wagons busy removing the excavated material. The grading was done by J. A. McGarry & Co., of Chicago. The machine was built by the Koehring Machine Co., of Milwaukee, Wis.

Stalactite Cave Moved from Arizona to New York—A beautiful stalactite-lined cavern was recently broken into at the Copper Queen mine at Bisbee, Ariz. The company placed the cavern at the disposal of Doctor Hovey, curator of geology at the New York Museum of Natural History, and he with three assistants, packed the cavern into 50 boxes and dispatched it to New York, where it has been reassembled. The age of the grotto is estimated at 25,000 years. One of these stalactites is 14 ft. high by 15 ft. in diameter at the base. Some of the stalactites and stalagmites removed weigh 800 lb.—"Engineering and Mining Journal"

Countersunk Rivets

By R. Fleming*

This article is the outcome of a research of the literature on the strength of countersunk rivets.

Much of the literature on riveted joints has been written from the standpoint of the shipbuilder and the boilermaker. Strength is given in terms of efficiency. By the efficiency of a joint is meant the ratio of the strength of a given width of joint to that of a solid plate of equal width. The structural engineer rarely uses the word "efficiency." He wants to know the shearing and bearing values of rivets so that he may put a sufficient number in his connections to take care of the stresses. These values are well defined for ordinary rivets. What value shall the structural engineer assign to countersunk rivets? He may read in an old Haswell,

> Countersunk riveting does not impair strength of joint as compared with external head.

and in the latest (1915) edition of Smith and Mark's "Machine Design,"

> Countersunk rivets make a much weaker and less reliable joint than the ordinary form, and should only be used when it is absolutely necessary that the surface of the plate be free from projections.

In his "Steel Construction" Burt writes

> It is to be noted that countersunk rivets are not as strong as rivets with button heads and are much more expensive, consequently they are not used unless absolutely required by the conditions.

The usual American practice for countersinking rivets in bridge and building work is to make the depth of head one-half the diameter of shank and the slope 30° from axis of shank. The practice is different in shipbuilding. For many years the English Lloyd's "Rules and Regulations" has contained the clause,

> The countersinking of the rivet holes is to extend through the whole thickness of the plate or angle when the thickness is less than 0.60 in., and when the thickness is 0.60 in. or above the countersinking is to extend through nine-tenths the thickness of the plate. The size of the countersink is to be in accordance with the figured dimension shown in Table 20.

The American Bureau of Shipping has the rule,

> The countersinking of all plates and angle bars to extend to within $\frac{1}{32}$ in. of being through the plate or angle bar.

The slope varies according to diameter of rivet.

Early Tests with Countersunk Rivets

The earliest recorded experiments on the strength of riveted joints were made in 1838 by Hodgkinson under the direction of Fairbairn. They are given in detail in a paper entitled "An Experimental Inquiry Into the Strength of Wrought-Iron Plates and Their Riveted Joints as Applied to Shipbuilding and Vessels Exposed to Severe Strains," by William Fairbairn, Esq., in the "Philosophical Transactions" of the Royal Society of London for 1850. This paper has had a remarkable influence in determining practice for riveted joints. Regarding countersunk rivets Fairbairn writes,

> The system of countersinking the rivets is only used when smooth surfaces are required; under other circumstances their introduction would not be desirable as they do not add to the strength of the joint but to a certain extent reduce it. This reduction is not observable in the experiments, but the simple fact of sinking the head of the rivet into the plate and cutting out a greater portion of metal must of necessity lessen its strength, and render it weaker than the plain joint with raised heads. This must appear evident from the fact of the sectional area of the plate being diminished, and the consequent reduction of the heads of the rivets, which in this state are less able to sustain the effects of an oblique or transverse strain.
>
> It is, however, satisfactory to observe that countersinking the heads of the rivets does not seriously injure the joint in its powers of resistance to a direct tensile force; but the rivets are liable to start when exposed to collisions or a strong impinging force, such as the sides of ships are frequently doomed to encounter.
>
> On referring to experiments (Table XI), the same results as to strength are obtained with the countersunk rivets as those with rounded heads; they are rather under the mean of the former experiments; but not more than is easily accounted for by the reduced section of the countersunk plates.

The experiments of "Table XI" were with $\frac{5}{8}$-in. rivets countersunk in plates 0.22 in. thick.

Masterly papers and reports on riveted joints, by Baron Clauzel, Professor Unwin and Professor Kennedy, may be found in the "Proceedings" of the Institution of Mechanical Engineers (London) for the years 1881 and 1885. An "Account of Some Tests of Riveted Joints for Boilerwork," by C. H. Moberby, may be found in Vol. LXIX (1882) of the "Proceedings" of the Institution of Civil Engineers. The tests were carried out by Kirkaldy. In Vol. XVI of the "Transactions" of the Institution of Civil Engineers of Ireland, Stoney has a valuable paper "On the Strength and Proportions of Riveted Joints" which he has partly embodied in his book "The Theory of Stresses in Girders and Similar Structures." Extensive tests of riveted joints have been made at the Watertown Arsenal and recorded in government reports; the index alone occupies several pages. Another series of tests of exceptional interest and value is reported in the 1905 "Proceedings" of the American Railway Engineering and Maintenance of Way Association. It is to be regretted that countersunk rivets are not mentioned in any of the foregoing papers.

Recent Tests Give Conflicting Results

In "The Strength of Plates and Rivets Used in Shipbuilding," by J. G. Wildish, "Transactions" of the Institution of Naval Architects, Vol. XXVI (1885), there are recorded some shearing tests made with $\frac{3}{4}$- and $\frac{7}{8}$-in. steel rivets in $\frac{1}{2}$-in. steel plates and 1-in. rivets in $\frac{3}{4}$-in. plates. With pan heads and countersunk points the $\frac{3}{4}$-in. rivets sheared at a pull of 11.160 to 12.500 tons per rivet, and with snap points at 10.825 to 11.049 tons. The $\frac{7}{8}$-in. rivets sheared at 14.910 to 15.792 tons with countersunk points and at 14.787 to 14.953 tons with snap points. The 1-in. rivets sheared at 19.475 to 21.986 tons with countersunk points and at 18.861 to 19.420 tons with snap points.

> Half of these tests had the countersinking stopped $\frac{1}{8}$ in. short of the full thickness of the plates, and in the other half the countersinking extended right through; but the results were on the whole as good one way as the other, and there was but little variation in them individually.

Further tests on the frictional resistance of riveted joints showed that taken altogether the friction was greatest for the countersunk rivets.

Textbooks on mechanics and strength of materials, a long array, are silent on the subject. It is taken up, however, in a few books on shipbuilding.

* American Bridge Co., 30 Church St., New York City.

"Shipbuilding in Iron and Steel," 1869, by E. J. Reed (afterward Sir Edward J. Reed), has a chapter, "Rivets and Rivet-Work," recording some tests of countersunk rivets. The remark

> It also becomes evident that countersunk riveting causes much less friction than the other systems. On comparison it will be seen that in nearly all cases steel plates and rivets give less friction than iron.

is interesting to read. In those days there was a distrust of steel rivets. Their use was prohibited by the Lloyd's Committee.

In his book, "The Design and Construction of Ships," 1908, Biles gives the results of a series of tests made at Glasgow University (probably under his direction). The ultimate shearing stress of $\frac{5}{16}$-in. countersunk rivets in $\frac{1}{8}$-in. plates ran from 14.63 to 20.83 tons per sq.in. while snap-headed and pan-headed rivets ran from 18.28 to 18.64 tons. The ultimate shearing stress of countersunk $\frac{3}{8}$-in. rivets in $\frac{1}{8}$-in. plates ran from 14.87 to 19.0 tons and pan-headed rivets ran from 17.27 to 18.73 tons.

Hovgaard has a chapter, Calculations for Riveted Joints, in his "Structural Design of Warships," 1915, in which the efficiency of joints with rivets countersunk according to Lloyd's rules is determined theoretically. The efficiency of plates and rivets is given in percentages. That these are of little value to the engineer in search of working stresses may be seen from the statement at the close of the chapter,

> There is in shipbuilding no established practice as regards the working stresses allowed on the rivets in the ship-girder; the complexity of the problem and the empirical nature of the strength calculations have so far prevented the adoption of any definite limits.

What Various Modern Specifications Prescribe

The majority of specifications for bridges and buildings give but little if any consideration to countersunk rivets. The Cooper "Specifications for Steel Highway and Electric Street Railway Bridges" reads,

> Rivets countersunk in material of less thickness than the diameter of the rivet, shall only be given a value compared to that of a regular rivet equal to the proportion of the thickness of the metal countersunk to the diameter of the rivet.

The specifications for highway and electric railway bridges compiled by the State Engineer of New York read,

> Rivets shall not be countersunk in any plate whose thickness is less than one-half the diameter of the rivet. In calculating the value of countersunk rivets the effective thickness of the plate which receives the countersunk head shall be considered as ¼ in. less than the actual thickness of the plate for ¾-in. rivets and as $\frac{5}{16}$ in. less for ⅞-in. rivets.

A number of railroad specifications make mention of countersunk rivets, though often only by a sentence like "Countersinking must be neatly done, all holes of the same size, and countersunk rivets must completely fill the holes." The Atlantic Coast Line specifies,

> Rivets countersunk on one or both heads will not be allowed except where they cannot be avoided.

The Queen & Crescent Route specifies,

> Rivets with ⅞-in. flat heads shall be preferred to countersunk rivets and countersunk rivets shall not be used in material less than $\frac{7}{16}$ in. thick. Countersunk rivets shall be considered to have but half the strength of round-head shop rivets.

The "General Specifications for Steel Structures" of the Southern Railway Co. contains the clause,

> No value will be given to ⅞-in. rivets countersunk in plates less than ⅝ in. thick, nor to ¾-in. rivets countersunk in plates less than ½ in. thick.

The Scranton & Binghamton Railroad Co. specifies,

> The effective bearing area of pins, bolts or rivets shall be the diameter multiplied by the thickness of the piece, except for countersunk rivets when half the countersink shall be omitted.

The Pennsylvania Railroad Co. in its "Standard Specifications for Steel Bridges" has the provision,

> Rivets with countersunk heads shall be assumed to have three-fourths the value of corresponding rivets with full heads.

The specifications of the Chesapeake & Ohio Railroad Co. contain the same clause. It is also found in a number of building specifications.

The New York Central Lines specify,

> Field-driven rivets, countersunk rivets and turned bolts shall be increased in number 25% over the number of full-headed shop-driven rivets required.
>
> The effective bearing area of a pin, a bolt or a rivet shall be its diameter multiplied by the thickness of the piece, except that for countersunk rivets half the depth of the countersink shall be omitted.

The Buffalo, Rochester & Pittsburgh Railway Co. and the Long Island Railroad Co. have the same provisions.

The "General Specifications for Railway Bridge Superstructure" of the Osborn Engineering Co. contains the clause,

> In deducting rivet holes to obtain the net section of riveted tension members, the rivet hole shall be taken with a diameter ⅛ in. larger than the undriven rivet for full heads, and ¼ in. larger for countersunk rivets in plates ⅝ in. or less in thickness.
>
> Where rivets are countersunk the bearing of the head shall not be counted.

Godfrey in his "Steel Designing," 1913, specifies,

> The shearing or bearing value of countersunk rivets in plates ⅜ in. thick and over will be taken as one-half the full value. Countersunk rivets in plates less than ⅜ in. thick will not be considered as taking any of the stress

The specifications of the Isthmian Canal Commission for the coaling plants at Cristobal and Balboa contain a clause reading,

> Countersunk rivets in metal ⅜ in. or less in thickness shall not be considered as resisting shearing or bearing. In metal ⅜ in. to ⅝ in. they shall be considered as having half value, and in thicker plates full value.

Waddell in his recent treatise, "Bridge Engineering," chapter LXVIII, specifies:

> Rivets with flat heads shall be preferred to countersunk rivets; the height or thickness of the flat head shall be three-eighths of an inch.
>
> In important members rivets shall not be countersunk in plates of thickness less than one-half of the diameter of the rivet. Rivets with flattened heads shall be assumed to have only eight-tenths of the strength of rivets that have full heads.

Rule Recommended by the Author

The rules in the specifications quoted must be followed until further experiments are made. The writer gives preference to the Pennsylvania Railroad Specifications modified to read as follows:

> Rivets with countersunk heads shall be assumed to have three-fourths the value of corresponding rivets with full heads, but no value shall be allowed for countersunk rivets in plates of a thickness less than one-half the diameter of the rivet.

For rivets with flat heads the writer suggests the following, being a slight modification of Waddell's specifications:

> Rivets with flattened heads of height not less than three-eighths of an inch, or one-half the diameter of the rivet for ⅝-in. rivets and less, shall be assumed to have nine-tenths the strength of rivets that have full heads.
>
> When heads are flattened to less than these heights they shall be assumed to have the strength of countersunk rivets.

Providing Water-Supply for a Large Militia Camp

SYNOPSIS—How the 22nd Engineers, without previous experience as a unit, laid out a 425-acre camp site, providing water and sanitary conveniences for 25,000 men, in 44 days.

The duties of military engineers on active service have been a prolific subject for speech and writing, both by men who know and by men who do not know. These duties for the New York Engineer Corps are exactly the same as those of the United States Army Engineer Corps, and their scope is stated with brevity and precision in the United States Army Regulations. One of these duties is to prepare camp sites for occupation by troops. How this particular and by by no means simple work is done was recently shown by a notable performance of the 22nd New York Engineers—Lieut.-Col. Eugene W. Van C. Lucas, commanding—which in little more than a month transformed an unimproved state tract into a healthful camp capable of supporting a full military division.

Military leaders are interested mainly in results, or at least that is the popular notion. They want a camp for some thousands of men ready by such and such a date. The task of providing this is put up to the chief engineer officer assigned to the division. How it is accomplished is the concern of the officer just mentioned and not of his superiors.

The point selected for the summer mobilization of the National Guard of New York was an 825-acre state tract located near the excessively rural community designated Beekman. Camp Whitman, as it was named, is adjacent to Green Haven, a stopping point daily for four trains of the Central New England Ry., which crosses the state land. Here, at Green Haven Station, the division quartermaster's depot was established. The town of Beacon, on the New York Central main line, is 17 mi. distant from the camp.

Colonel Lucas had been ordered on May 16 to have the camp ready for occupation by July 9, and on May 19 a detachment of 12 men and two officers came to Green Haven and established Camp Whitman. On June 18 came the President's order for instant mobilization of the National Guard, pushing the date of occupancy forward three weeks and increasing the engineering difficulties by several hundred per cent. By June 30 the water-supply was available, and a number of the units were encamped.

The necessary work divided naturally into four classes: (1) Camp survey for determining regimental locations and for laying out systems for water-supply and roads; (2) the development of an adequate water-supply—the most important and absolutely essential work of all; (3) the development and construction of an adequate road system; and (4) the reconnoissance and mapping of 61 sq.mi. of adjacent farmland to be used for maneuver work during the proposed encampment. Many minor problems had also to be taken up and disposed of.

Camp Survey for Regimental Location

The camp survey was worked out by Lieutenant Stockwell and consisted of a stadia survey with 10-ft. contours, plotted to a scale of 200 ft. to the inch. Lieutenant Stockwell's work was completed in a thorough and effective manner, as was demonstrated on several occasions when organizations arriving at the camp without previous notice were promptly located.

The object of this survey was to determine a sufficient area for all the organizations of the division—each regimental camp to be situated on land free from woods

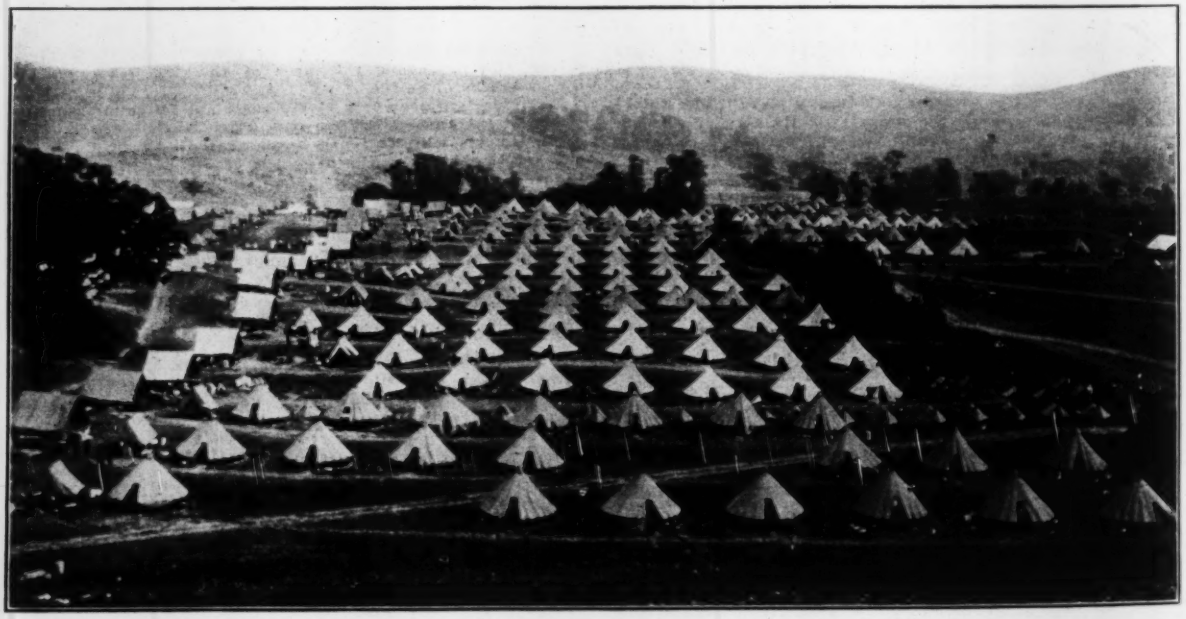


FIG. 1. ENCAMPMENTS OF 10TH AND 65TH INFANTRY AT CAMP WHITMAN, N. Y.

of swamp and below the ridge on which the water tanks were to be built. The survey covered an area of 425 acres and served as a general working drawing on which the locations for each regiment were made. This map was also used for laying out the communications generally and for working out the water-supply arrangements.

Three Water-Supply Systems Available

To obtain water for the camp, it was necessary either to pipe it from Fishkill Creek or to sink wells. The water of Fishkill Creek is used for the water-supply of Beacon, located at the confluence of the creek with the Hudson. It was not deemed advisable, however, to trust to surface water for a camp of from 15,000 to 20,000 men; the chances of pollution were too numerous. Decision was therefore made in favor of wells. The work was outlined on May 30, and on June 7 the orders were placed for the necessary materials for pumping station, supply line and tanks. The time between these dates was occupied in meeting necessary requirements, such as calling for bids and opening them. Previous to June 16, material orders were placed by the chief quartermaster, but after that date by the chief engineer.

The services of two local well diggers were secured, and water deposits were found at two localities—one a short distance north of the state road near Green Haven crossroads and the other near the western boundary of the tract, about 300 yd. south of the state road. When first found both deposits were of equal promise, but the second did not develop a sufficient quantity of water and was abandoned. The former gave better results and was developed by Lieutenant O'Connor until a daily supply was obtained of about 200,000 gal. from five or six wells.

This supply was brought into service on the morning of June 29 and constituted the main source for about 10 days. After a week's use the gravel pocket had been cleared of fine sand and the pipe system of grease, etc., and the water was officially pronounced fit to drink. This source of supply was never regarded as a satisfactory primary source and was replaced by the Carter group of wells, described hereafter, as soon as the latter

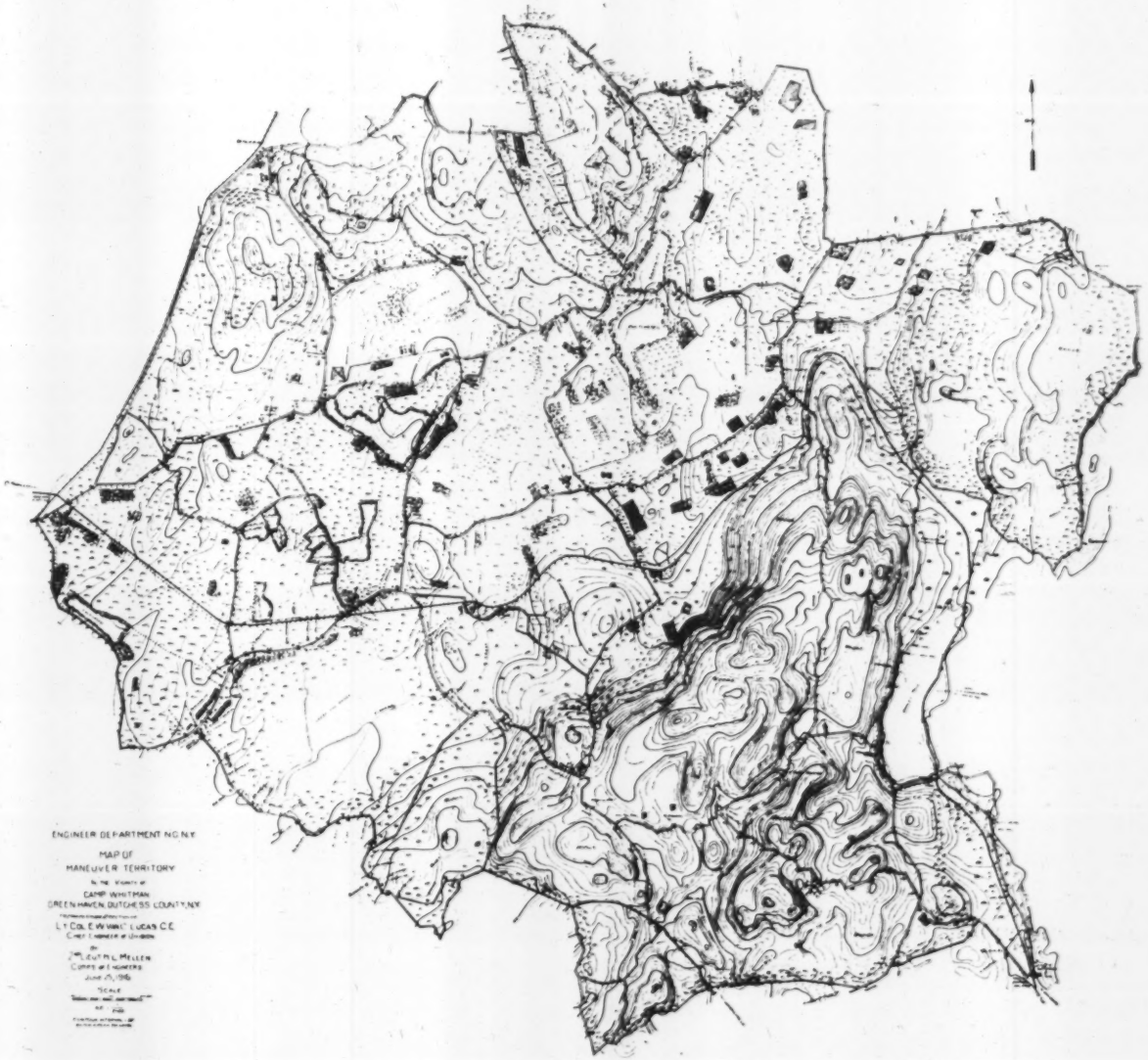


FIG. 2. RECONNAISSANCE MAP MADE IN THE FIELD BY THE 22ND ENGINEERS OF NEW YORK

were available. The O'Connor group then became a first reserve supply. The wells of this group were about 35 ft. deep, of 6-in. casing, with a 4-in. pipe inside. The inner pipe extended down to within 1 ft. of the bottom of the casing. No strainers were used, the sand dropping into a pipe sump that had to be cleaned several times daily. The wells were connected together and worked by a plunger pump of Fairbanks-Morse manufacture, which had a capacity of 250 gal. per min. It was driven by a 10-hp. single-cylinder 4-cycle gasoline engine of the same make. At full capacity, the wells had dried.

Meanwhile an offer had been received from Ralph B. Carter, of the Ralph B. Carter Co., of New York City,

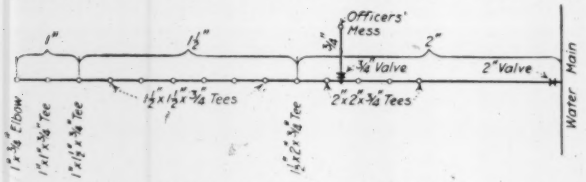


FIG. 3. REGIMENTAL WATER-SUPPLY LINE

to make a systematic exploration of the subsurface water possibilities of the whole camp site, the method being a series of 2-in. wash borings put down at the rate of about three daily. The fifth boring, made by Mr. Carter on the night of June 27, discovered the water deposit later developed as the primary supply for the camp—a series of flowing wells tapped at a depth of 45 to 50 ft. below about 30 ft. of clay and pronounced by the experts of the State Department of Health an absolutely pure water. The capacity of this deposit was estimated at 1,000,000 gal. daily and was utilized to the extent of about 250,000 gal. daily, by means of a gang of 15 to 20 two-inch wells connected at the top.

Before the development of the first water-supply and just after the President's call for the National Guard the demand for an adequate water-supply became so acute that it was not deemed wise to trust entirely to a successful development of a subsurface water-supply, which is necessarily a slow process. Accordingly, while the subsurface work was continued, it was determined, on the morning of June 22, to bring in also as a final reserve the water resources of Fishkill Creek, estimated to provide a minimum supply of over 2,000,000 gal. daily. For this surface supply, with the approval of the representatives of the State Department of Health, a chlorination plant was installed at the pumping station on Fishkill Creek. It was never necessary to utilize the Fishkill Creek supply, but as the project stands there are available three sources—first, the Carter development, brought in on June 28; second, the O'Connor group of six wells first used; and finally, the surface supply from Fishkill Creek.

A 50,000-Gal. Wood-Stave Tank Erected Overnight

Mr. Carter made some further exploration and discovered an additional deposit equally as good as the primary supply and located on the western edge of the state land near the railroad track. It is estimated that a sufficient water-supply for 100,000 to 200,000 men can readily be obtained at this location. Water requirements are estimated on the basis of 20 gal. per man per day.

The project included the erection of two 50,000-gal. tanks, of which only one was built. It is a wood-stave tank 20 ft. high by 22 ft. in diameter. The site decided upon for it is on the top of a hill, requiring a lift of 160 ft. from the water main. The foundations consist of four concrete piers 4½ ft. deep, 20 in. wide at the bottom and 18 in. at the top. Concrete for the first pier was mixed by hand, native sand and gravel of fairly good quality being used. A pat test of this concrete gave poor results. Foraging unearthed a small tumbler, or hand mixer, owned by a farmer, about 8 mi. distant from the site. This was obtained, repaired by the local blacksmith, belted to a 4½-hp. gasoline engine, and turned out 20 batches an hour. The foundations were placed by a gang of 12 men. Foundations were also placed for the second 50,000-gal. tank.

In the course of the foundation work, material for the wood tank arrived and was stored on a hill near-by. With apparently unskilled labor the tank was erected in 36 hr., the last 16 of which were consecutive and at night. This work included building a scaffold for putting on the hoops, which came four sections to the hoop. Power lights, or gasoline flares, were used at night. Corporal Fogarty, a former chief petty officer in the United States Navy, was in direct charge of the erection, under the supervision of Lieutenant Bregenzer.

The main and distributing system aggregates more than 10 mi. of piping. The mains are 6-in. "Universal"

FIG. 4. LAYOUT OF WATER MAINS AT CAMP WHITMAN

pipe, furnished by the Central Foundry Co., of Newark, N. J., while the laterals are 4-in. "Universal" and 4-in. wrought iron. The distributing lines vary from this size to ¾ in. The piping for the most part is laid in trenches and has about 1 ft. of cover. A 3-in. line to the base hospital and 69th Infantry was an emergency job and was laid on top of the ground. It is over 2,000 ft. long and was laid complete in one afternoon by working parties from companies A, B and D, 22nd Engineers.

Fig. 3 shows the water requirements for a regiment in camp. Inside the area occupied by the unit there must be fifteen 1-in. faucets, so that each company has a faucet; and there is one for the machine-gun detachment and another for the kitchen. There is a faucet for the noncommissioned staff and hospital corps, and one for officers. Each regiment is provided with a shower-bath house having 18 outlets.

The water-supply system was laid out by Lieut. J. E. Baker, and its installation was under his immediate supervision as special assistant to Colonel Lucas. It was through Lieutenant Baker that the valuable coöperation was secured of Westinghouse Church Kerr & Co., by whom he is employed in civil life.

Lieut. M. H. Gray rendered important service as quartermaster and in many special ways, being the only officer besides Colonel Lucas who was connected with the work from start to finish.

Time Only for Emergency Roadwork

Previous to the arrival on June 20 and 21 of the 22nd Engineers as an organization the available force (15 to 50 men) was too small to undertake roadwork; but on June 22 the roadwork was assigned to the first battalion, and a good start had been made when, on June 24, the first battalion was ordered mustered into the United States service. Thereafter it was not possible for the regiment to do any further roadwork other than of an emergency nature, as its entire available energy was required on the development of the water-supply system up to the time of its muster into the United States service and departure for the Mexican border.

While there was little opportunity for road building or repair, there was a certain amount of widening to be done before freight could be passed over the road from the railway station. One of these jobs was completed from the Green Haven post office toward the station—a distance of 400 ft.—in 1½ days. The principal requirement of such work is adequate drainage.

The work was done by hand, the tool ratio being 2 shovels to 1 pick, a gang of 100 men being used. Among the men were discovered two highway inspectors, and these were placed in charge of sections, the corporals handling the subgangs. The work was carried on by three classes of men. The excavating gang constructed side ditch and shoulder, while rock layers lifted stones from a wall along the road and placed the larger of these as edging. The smaller stones were used to build up the road and shoulder and to line the ditch, thus providing blind drainage. A casting gang disposed of the excavated material, which, being principally of sand and gravel, was used to fill the interstices in the stones and as a preliminary binder. Two gravel teams spread gravel to a thickness of 2 in. over the top.

Reconnaissance Work

On June 4, four field parties were organized, each comprising a topographer and an assistant. The function of these parties was to make a reconnaissance survey for the control of the elaborate topographic map, reproduced herewith (Fig. 2). In assigning daily work to the parties the maps of the United States Geological Survey were found extremely useful, but their scale was too small to allow them to serve as military maps. The control map was constructed on a scale of 3 in. = 1 mi.,

LIEUT.-COL. E. W. VAN C. LUCAS

and showed roads, fields and houses. The plane-table method was used, and the work went on rain or shine, water having no injurious effect upon the thin sheets of celluloid used in place of sheets of paper on the sketching boards in wet weather. The topography of Camp Whitman and vicinity is uneven; there are mountains, plains, woods and farmhouses. The topographic party taking the mountain section used an aneroid; the other parties took elevations with tapes and a balanced clinometer. The work of the day was closed in every night on the control. Corp. F. O'Brien deserves special mention as he made the high record, sketching 12 mi. of road in one day. The control points were along the road radiating from Green Haven as a center. The control map was finished in five days, of which four were rainy. The work was organized and conducted by Lieut. C. E. Bregenzer.

Lieut. H. L. Mellen took hold of the work on June 13, filling in the topographical data, shown in Fig. 2. This work proceeded for six days without any special haste, when the rush order came. At once 15 to 20 parties were sent out, and the entire 64-sq.mi. area had been thoroughly mapped in four days.

The map was finished in five days, requiring the equivalent of 72 parties—that is, 18 parties per day for four days and one day's work in the office at the end.

The work of the engineers also included laying out the necessary spur tracks, three in number, for the service of the quartermaster's depot at Green Haven station. This layout was made by Lieut. M. H. Gray, working with the engineers of the Central New England

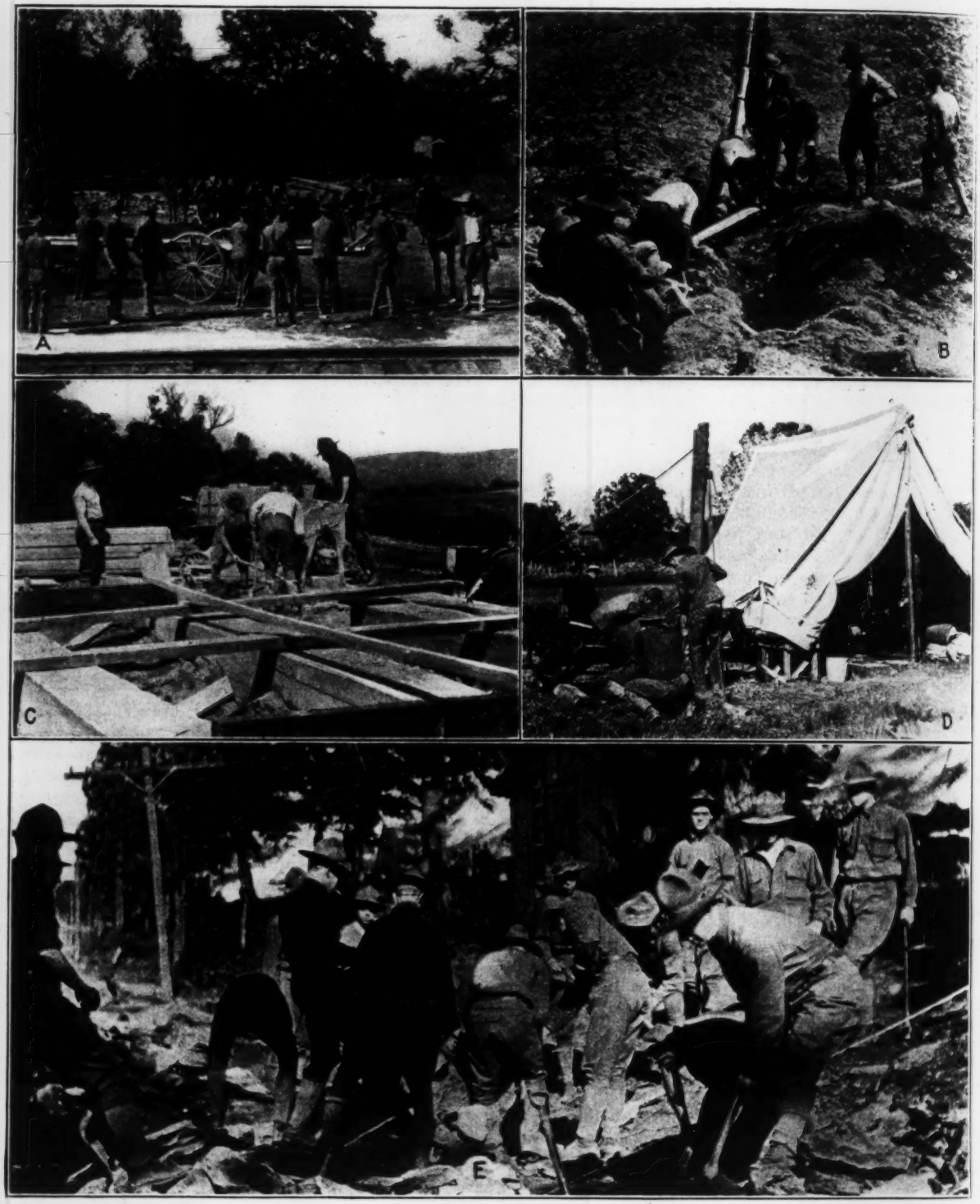


FIG. 5. 22ND ENGINEERS PREPARING WATER-SUPPLY AND ROADS AT CAMP WHITMAN, N. Y.

A—Loading pipe for water mains. B—Laying 6-in. "Universal" in shallow ditch. C—Building foundations for 50,000-gal. wood-stave tank; hand mixer at work. D—Pumping plant of 6-well secondary supply. E—Repairing road, using stones from an adjacent wall

Ry., and the work itself was done by the railway company. Deliveries of material were at first slow, but after the working organization had been established and the placing of orders taken over from the Quartermaster Corps by the Engineers, the reverse was the case. On one occasion 2,500 ft. of 6-in. pipe arrived from Newark 24½ hr. after placing the order by telephone, and in another case a 60-hp. gasoline engine and a two-stage centrifugal pump were received from Aurora, Ill., by way of Chicago, in 30 hr.

The actual work of preparation of the camp site began about May 22, after the establishment of the camp, and for a month the detachment was small (15 to 50 men) and its personnel constantly changing. After June 21 the entire 22nd Engineers was available until the muster-in of the battalions, and assistance was also furnished by the 69th Infantry and the 2nd Infantry. As a rule, the engineers carried the water mains to each camp, and the interior piping was laid by each regiment for itself.

Material assistance was furnished by the State Department of Health through Chief Engineer Horton, Assistant Engineers Sanborn and Chase and Dr. Van Winkle; by the State Department of Highways through Assistant Engineers Waite, Bixby and Howe.

Goose Lake Valley Irrigation System, Oregon

By Kenneth A. Heron*

The Goose Lake Valley in south central Oregon consists of about 60,000 acres of agricultural lands surrounded on the east, north and west by the high Sierra Nevada range and foothills. Originally covered with a thick growth of sagebrush, in later years a small area has been cleared for farming purposes and a limited portion of this, in wild hay, has been irrigated from the several streams flowing to the lake.

Seven years ago the Oregon Valley Land Co. projected a reservoir and canal system that would supply water for irrigation purposes to practically all the irrigable lands in the valley, and construction that has since been accomplished by that company and its successor, the Goose Lake Valley Irrigation Co., has advanced to a stage that insures its completion in the near future. The system consists of a reservoir formed by a dam on Drews Creek, the North Drews canal, the South Drews canal and various laterals.

During a period of 15 yr. the annual precipitation has not averaged more than 14 in. Of this 80% falls in September to May, when of little value to growing crops.

North Drews Canal and Structures

Beginning just below the Drews Dam (Fig. 1) the North Drews canal runs along the steep cañon sides 4 mi. before it reaches the west edge of the valley and turns north and skirts the foothills for 30 mi. A proposed extension is shown on the map.

The reservoir outlet tunnel is located on the south side of the creek about 16 ft. above its bed. The canal is carried over the cañon by a 12x7-ft. timber flume with a capacity of over 700 sec.-ft. After leaving this flume the canal is located in two places through an earth and gravel formation for short distances, and for the remainder of the distance along the cañon side the canal is carried through rock cuts and timber flumes set on a wide shelf cut out of the rock. Five of these flumes are built. One of them is shown by the view, Fig. 2. To avoid a long solid rock cut in one place tunneling was resorted to for a short distance at a point about 2 mi. below the dam. After leaving the cañon there

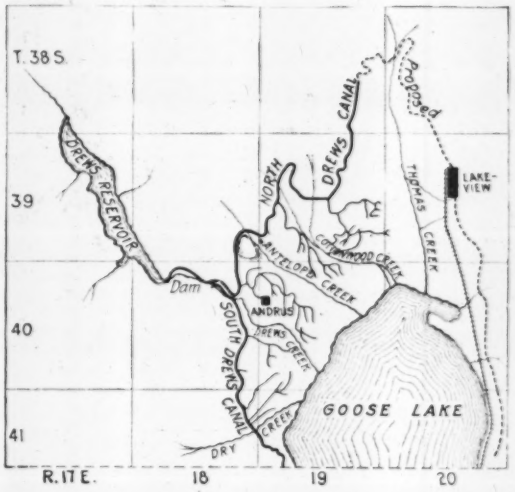


FIG. 1. MAP OF GOOSE LAKE VALLEY IRRIGATION SYSTEM, OREGON

are no structures of importance for 7 mi. Here another large timber flume is employed to avoid the necessity of a long canal through difficult material. Flume 600 (Fig. 3) as this is called is 8x5½ ft. in section and 900 ft. in length.

Beyond this point the canal capacity decreases gradually and a number of semicircular steel flumes (Figs. 3 and 4) are constructed. The largest of these crosses Cottonwood Creek and is 9.6 ft. in diameter and 2,100 ft. in length. It is supported on piling and framework of heavy timbers. All of the steel flumes on the system have flaring intakes of flume metal and concrete outlet structures.

A timber-lined tunnel 900 ft. long just west of Cottonwood Creek was built instead of a 2½-mi. canal around a high ridge. (See Fig. 3.)

South Drews Canal and Laterals

On account of the topography of that portion of the valley south of Drews Creek the elevation of the south canal is about 75 ft. below the north canal and the former heads on Drews Creek about 2 mi. below the dam. It follows the creek on the north side to the point where the higher canal leaves the cañon and then turns in the opposite direction (Fig. 1.) The canal consists of sections of timber flumes, solid rock cuts, masonry walls and earth and gravel cuts. Where the canal turns south and crosses Drews Creek a semicircular steel flume 6.4 ft. in diameter and 575 ft. long, on a wooden substructure, has been built. Seven other creeks and depressions are crossed by flumes of this kind.

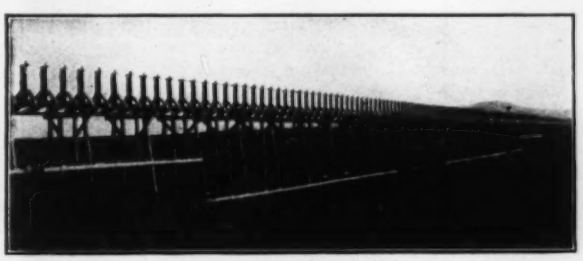


FIG. 2. TIMBER FLUME, NORTH DREWS CANAL

*Chief Engineer, Goose Lake Valley Irrigation Co., Lakeview, Ore.

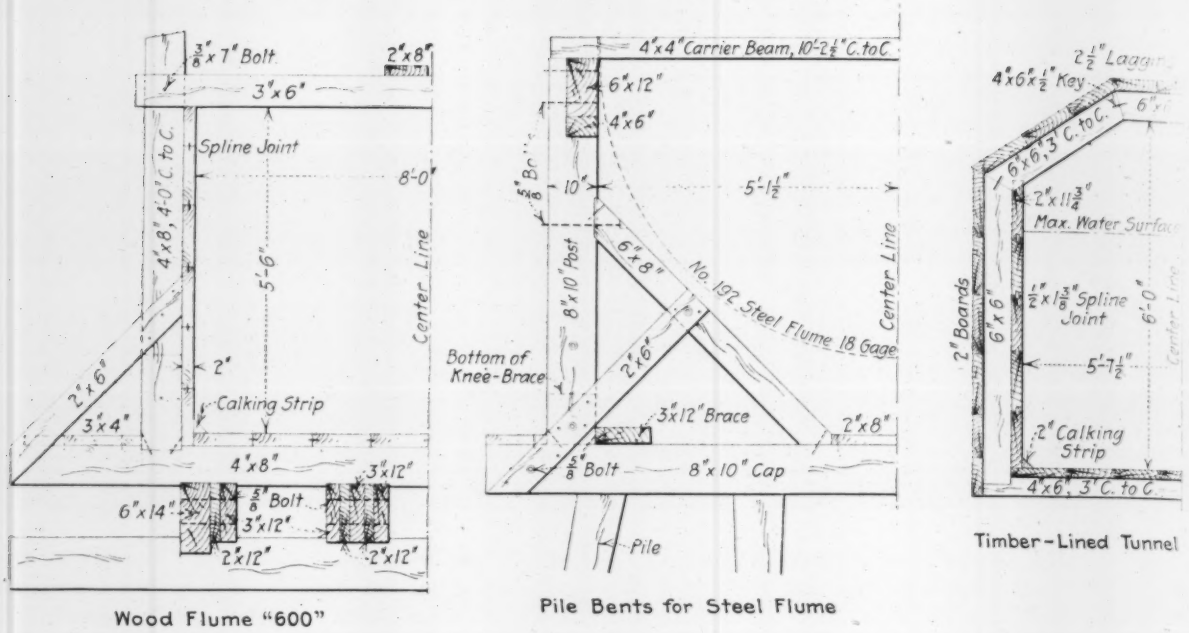


FIG. 3. DETAILS OF WOOD FLUME, STEEL FLUME ON TIMBER BENTS AND TIMBER-LINED TUNNEL, NORTH DREWS CANAL

Work has been in progress on the main canals since 1910. In 1914 the north canal was completed and in 1915 water was first turned into the south canal.

Prior to the summer of 1915 only two short laterals had been constructed for distribution purposes. Since then main laterals have been located to irrigate all of the territory on the west side of the lake and during the spring of 1916 construction was pushed as fast as possible in order to deliver water to as large an acreage as can be irrigated this season. It is hoped that by fall all parts of this area will be within striking distance of a lateral.

All the timber used for the dam, flumes and other structures on the canals has been milled by the company at its mill near the Drews Dam. On account of poor facilities for bringing in material from Alturas, Calif., a distance of 65 mi., at the time most of the work was done materials for construction purposes were limited to timber and cement at a very high price. Later the Nevada, California & Oregon Ry. was extended to Lakeview from Alturas, and this has made possible the use of such other materials as are necessary.

The company's own telephone lines extend to the principal points on its irrigation system. Most of the lines consist of No. 9 galvanized wire strung on poles 175 ft. c. to c. Eight-foot cedar stubs are set from 3 to 4 ft. in the ground and 18-ft. pine posts are wired to these.

FIG. 4. COTTONWOOD CREEK STEEL FLUME

New Boulevard Overlooking San Francisco

The new Twin Peaks Boulevard in San Francisco, from a scenic standpoint the most notable road on the San Francisco peninsula, is now rapidly nearing completion. It starts at the intersection of St. Germain and Burnett Ave., ascends and encircles near their summits the two hills known as Twin Peaks, at an elevation of 825 ft., and thence descends to terminate in Corbett Ave. at a point about 900 ft. distant from the westerly boundary line of the San Miguel Rancho.

The roadway consists of an asphalt pavement 25 ft. wide, with a 7 ft. 6 in. rock shoulder adjoining it on each side, giving a total width of 40 ft. The pavement is composed of a concrete base 6 in. in thickness, covered by a binder course 1½ in. thick and a 1 in. asphaltic wearing surface. Before constructing any pavement, the subgrade was thoroughly compacted by rolling with a 12-ton road roller.

Surface drainage is carried off by 12-in. corrugated steel culverts, encased in concrete, underlying the roadway at required points. Water collecting in side ditches is discharged into concrete inlets and thence through the culverts. A guard rail, consisting of two 2x6-in. surfaced pine rails nailed to 6x6-in. surfaced redwood posts 8 ft. apart was constructed in the shoulder adjoining the fill side of the roadway.

The contract for the construction of that section of the boulevard extending from St. Germain Ave. through the City Reservoir site was awarded in 1915, to Eaton & Smith, for the estimated sum of $24,058. This portion of the boulevard is approximately 2,800 ft. long and

FIG. 1. LOOKING NORTH ON TWIN PEAKS BOULEVARD, SAN FRANCISCO. RESERVOIR IN FOREGROUND, RICHMOND DISTRICT AND PRESIDIO IN DISTANCE

its construction necessitated the excavation by steam shovel of approximately 30,000 cu.yd. of rock and earth, the construction of 70,000 sq.ft. of pavement and 2,730 lin.ft. of guard rail.

The maximum grade on this section of the boulevard is 9% and the sharpest curve has a radius of 60 ft. The roadway on curves is superelevated to insure safe and easy riding, the maximum superelevation on the curve mentioned being 14 in.

One of the most notable features of the boulevard is a curve forming a full semicircle or horseshoe, 3,173 ft. in length, with a center line radius of 68 ft. To eliminate accidents on this curve, 3,000 cu.yd. of rock were excavated in the interior core within the horseshoe to give a clear and unobstructed view across same.

The contract for the second section of the Twin Peaks Boulevard, extending from the City Reservoir site to Corbett Ave., was awarded on Sept. 20, 1915, to F. R. Ritchie & Co., for the estimated sum of $54,745. This unit of the boulevard is about 7,900 ft. long, 6,821 ft. of this being built on an acquired right of way. The contract included the excavation of approximately 6,300 cu.yd. of rock and earth and the construction of 202,121 sq.ft. of pavement, 960 lin.ft. of 12-in. culvert and 8,000 lin.ft. of guard rail.

Excavation was performed by a steam shovel. For fills the excavated material was placed in the piles by means of scrapers and dump wagons and thence rolled in layers by a 12-ton road roller. One of the fills underlying this roadway is 60 ft. deep. The surface of the side hills underlying the fills was thoroughly plowed before placing any material for fill.

An unsurpassed view of the City of San Francisco, the Pacific Ocean, Bay, Marin and Alameda County hills is obtainable from the summit of the boulevard.

The work has been done under the direction of M. M. O'Shaughnessy, City Engineer of San Francisco.

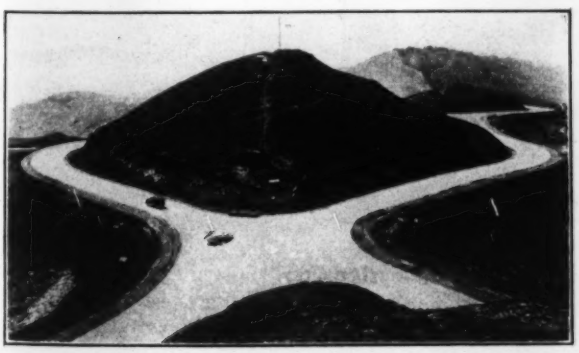


FIG. 2. LOOKING SOUTH ON FIGURE EIGHT SECTION OF TWIN PEAKS BOULEVARD

Widening Japanese Railways to standard gage has recently been announced in press dispatches as likely soon to be undertaken. The basis for this report is probably a recent public speech of the Premier of Japan, in which he announced himself as strongly in favor of the change of gage. From official sources "Engineering News" has ascertained that no early action in the matter is likely. The subject is still under discussion pro and con, as it has been for half a dozen years past, and the Japanese Parliament has made no appropriation for the work. The Imperial Railway Bureau, which operates the government railways of Japan, has been urging the widening of the gage to standard for six years past. In building all new bridges, tunnels, etc., during that time it has made them wide enough for standard-gage rolling stock. The large expense involved in the change of gage, however, causes widespread opposition to the plan, its opponents urging that the increased traffic could be more economically handled by steamers than by rail, nearly all the important cities of the Empire being on tidewater. They also urge that if a change of gage were made it would be necessary at the same time to expend a large amount in cutting down grades and reducing curvature. The difficulty in raising capital on account of war conditions and the high prices of engineering materials, rails, etc., will have its part in delaying any definite action on the change of gage.

Field and Office

Contractor talks on night work—Heaviest concrete girder in Philadelphia poured in three lifts—Pulling sheetpiles with locomotive crane and with hammer—Two-ton traction derrick weighing but 10 tons—World's tallest piledriver

Cost of Night and Day Work

By H. B. Whitney*

The editorial in *Engineering News* of Aug. 17 on comparative cost of night and day work brings to mind the experience of 30 years on double shift, covering several kinds of construction work. I was employed on river and harbor-improvement work for eight years, operating single shift, and was a licensed master of steam vessels for the Mississippi River and tributaries, and a pilot on the upper Mississippi River. Thus I had some acquaintance with night work through piloting, and was versed in the difficulties of that kind of work before I was called upon to operate day and night continuously.

My first experience was during 1886 in connection with dredging for the United States Government, as captain and pilot of a towboat handling scows. I expected to do nearly as well at night as on the day shift; but I found that the nights were more calm, and every month averaged a small per cent. better at night than by day, when high winds caused slow handling of the scows.

On dam and shore-protection work the force was on duty during the day only. Nights were devoted to delivering the empty barges at various quarries and brush camps and bringing loaded material to the dam. The day shift handled short tows and delivered barges to and from the dam. We always got as good service from the night work as from the day work, but arranged our schedule to suit the conditions.

I have handled betterment work on railways with double shift, and the same conditions exist; that is, the work must be picked for the night crew. There is no profit in night work unless the conditions are such that the night force can be favored. For opening up a borrow pit and fill, plans should be so made that the night force will have a clean dump and small crew. Then 100% efficiency can be secured. The day force will raise track and do all work requiring hard labor.

On drainage work, where kerosene lamps and torches were used we usually averaged 80% efficiency at night. With electric lights there is very little difference in the two shifts, provided the day shift is careful about making repairs, so that the night crew will not meet with accidents from breaks caused by not being able to see loose bolts, etc. I have before me the daily reports of a dredge operating day and night. The record shows the following progress in length of ditch:

Aug.	Day, Ft.	Night, Ft.	Aug.	Day, Ft.	Night, Ft.
1	190	150	7	160	140
2	190	190	8	100	150
3	190	200	9	30	110
4	190	150	10	160	300
5	150	150	11	160	210
6 (Sunday; crews changed shifts)					

The day crew consists of three men, besides one to boat coal and a cook. The night crew has three only, the coal man and cook not being necessary. The day crew attends to repairs and all extra work. These figures show that the day crew cut 1,520 ft. of ditch and the night crew 1,750 ft. The ditch is 6 ft. wide in the bottom and from 7 to 13 ft. deep, with 1 to 1 slopes. The 300-ft. run of the night crew was cleaning up the bottom, the day crew having moved the dredge back for them.

My experience is that a man cannot stand up to night work and perform hard labor and give the value on his services that he can on day labor. Bright, active men can give 100% efficiency at night if the work is properly laid out for them. My steam shovels gave just as good results at night as by day. Train service was about 90% value at night. The dump gang was just sufficient to dump from trestle or raised track. I avoided all shoveling as far as possible.

A crew working single shift will generally handle more than half the material that a double crew can put out, but with overhead expense and other incidentals the cost per cubic yard excavated is less with double crew than with single crew. Every job has its own peculiarities and its own average of relative cost of night and day work. The work mentioned above is from a 1-yd. dredge. I find better results from night crews on larger dredges. Much more depends on the comforts furnished a night crew than a day crew. They must be well fed and have good sleep to get high efficiency. A contractor at or near a city loses control of this feature if the night crew board themselves. But night work pays if taken in hand properly and if conditions are fair, and it pays better if favored by the plan of work giving the difficult part to the day shift.

*Contractor, Emmetsburg, Iowa.

Deep Concrete Girder Poured in Three Lifts

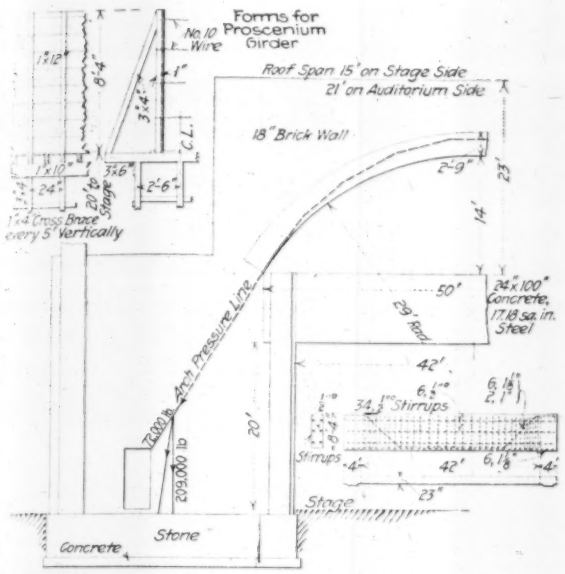
By Louis Muhlhausen*

A 57-ton concrete girder spanning 42 ft. has been built in the new Stratford Theater, Germantown Ave. and Venango St., Philadelphia, to span the proscenium opening. The girder is 24 in. wide by 100 in. deep. It was built by Ketcham & McQuade, contractors, of Philadelphia.

The reinforcement comprises twelve 1⅛-in. square rib bars (half of them bent) and two 1-in. square rib bars, bent. The vertical reinforcement comprises 34 ½-in. round stirrups. There are also six ½-in. square rib bars as horizontal side reinforcement, three on each side in the depth of the girder. Slag concrete was used for the girder.

This construction is especially interesting because it is the first reinforced-concrete proscenium girder in any Philadelphia theater, all others being steel plate girders fireproofed with concrete. Its weight exceeds by nine tons any existing concrete girder in the city.

*Engineer to Ketcham & McQuade, Contractors, 1029 Brown St., Philadelphia, Penn.



LARGE CONCRETE PROSCENIUM GIRDER, AND RELIEVING ARCH OVER

A relieving arch had to be built over the girder under the provisions of the Philadelphia building law. This is a plain brick arch, but it was complicated a little by the fact that the line of action had to avoid the stage door. The arch was made of such rise that the weight of the blank wall between proscenium opening and stage door brings the line of thrust down and keeps it in the solid brickwork at a safe distance from the opening.

A plain box form was built, as sketched in the drawing. The stirrups were placed in the form, then two of the straight 1⅛-in. bars and these wired to all stirrups. The other four straight bars were placed loose. Separators 4 ft. apart, each a short length of 1⅛-in. bar, were laid across this first layer, and the bent bars laid on them, the two outer ones being wired to the stirrups. The ⅞-in. side bars were then also wired to the stirrups.

Pouring the concrete was done in three installments. A tower about 100 ft. away spouted the concrete directly into the form. The first few batches were made with fine slag—about ⅜-in.—and were mixed very wet. After the top layer of the tension steel was covered, the slag was changed to ¾-in. material. The pouring continued to a depth of 3 ft.

At this time the concrete was diverted to another part of the roof, so as to allow the 3 ft. of concrete in the girder to set partially. Then the second lift was added, and after a similar interval the third. The one day's work completed the girder and the adjacent roof sections.

Contractor's Traction Derrick

A traction derrick has been built that will find employment by contractors of almost every description. It weighs 19,500 lb., has a lifting capacity of 4,000 lb. and dimensions that bring it within railway and highway clearances so that it will travel almost anywhere a motor truck can go. In addition to its work as a derrick it can be equipped with a bucket for excavating or unloading and can spot cars by means of a winch-head.

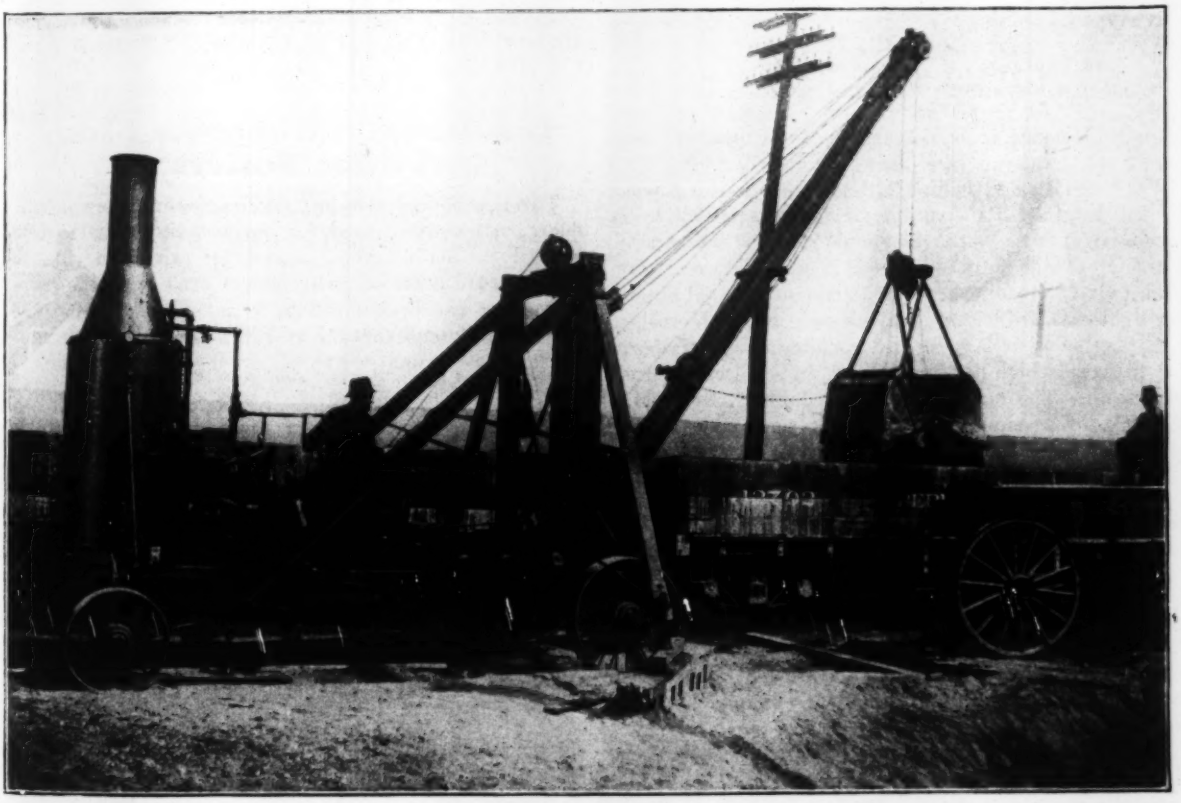


FIG. 1. BYERS' "AUTO-CRANE" ON A MUNICIPAL ROAD JOB AT FLINT, MICH.

The four-wheeled car body is 17 ft. 4 in. long by 6½ ft. wide, with a 15-ft. 4-in. wheelbase and a height from ground to top of A-frame of 12 ft. Either a ½-yd. clamshell or a 12-ft. orangepeel can be used at the end of a 20-ft. boom.

The forward or propelling wheels are 38 in. in diameter with 10-in. face and ¾-in. tires. On these wheels are sprockets driven by a heavy steel pintle chain. There is also a differential. The steering wheels are 30 in. in diameter and are mounted on a pivoted axle and steer by handwheel and worm. An emergency hand brake is provided. The wheels are drilled for attaching spuds, if desired. The propelling speed forward or reverse is 200 ft. per min.

The machine is built with three power friction drums for use with clamshell or orangepeel. Where the outfit is used only as a derrick, the same frame is employed with but two power drums, space being provided for a third drum, so that the extra drum can be added in the field, if

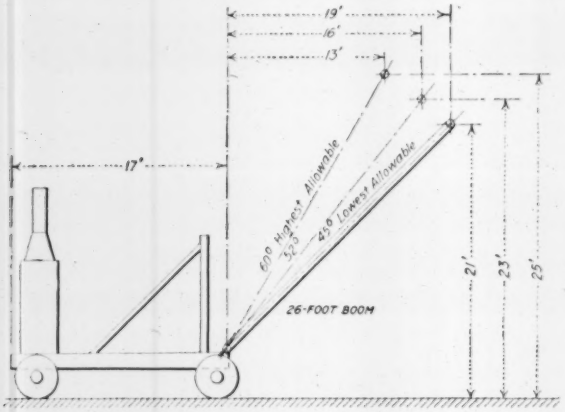


FIG. 2. BOOM HEIGHT AND RADIUS AT DIFFERENT ANGLES OF BOOM OF BYERS' "AUTO-CRANE"

desired. Housing is provided in the form of an all-steel cab.

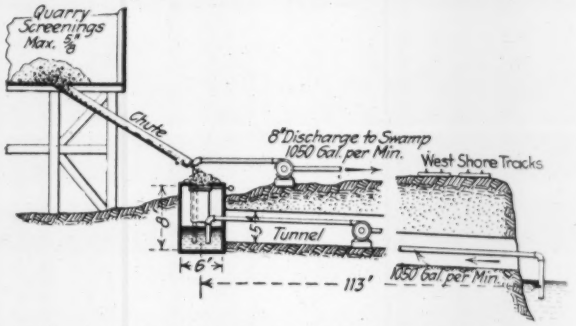
The boom is an 8x10 stick having a three-rod steel truss the full length. If the contractor desires a bucket to be furnished with the outfit, he should describe the general character of the work on which it will be used. For sand, gravel, screenings and similar loose material almost any light standard bucket is satisfactory, but with crushed rock, slag or other large or heavy materials it is important to have a high-powered bucket, in some cases fitted with teeth. The manufacturer, the John F. Byers Machine Co., of Ravenna, Ohio, guarantees one round trip per minute of boom and bucket.

Centrifugal Pumps Dispose of Quarry Waste

The disposal of large amounts of quarry screenings (⅝ in. and smaller) constitutes a troublesome problem at a number of crushing plants. The Upper Hudson Stone Co., Marlborough, N. Y., has devised an unusual method for wasting this material. It is pumped from a sump to marsh lands.

The plant is situated on the west bank of the Hudson River, its screenings storage being not more than 150 ft. from the water. Under the tracks of the West Shore R.R. and extending from the river to the plant is an old tunnel about 5x5 ft. in cross-section. A centrifugal pump was placed in this tunnel, as shown in the sketch, drawing water from the river and discharging into a covered sump or cistern, 6x6 ft. in plan by 8 ft. deep. The waste screenings are chuted to the top of this cistern, being dumped inside through a sliding trap controlled by a handle, as illustrated. When the door is slid open, the screenings fall through a canvas bag into the water, as shown. This arrangement, in addition to eliminating the usual dust, is also quite flexible.

The mixture in the cistern contains about 15% solids and is pumped out and wasted by means of a second pump



HOW QUARRY SCREENINGS ARE WASTED AT UPPER HUDSON PLANT

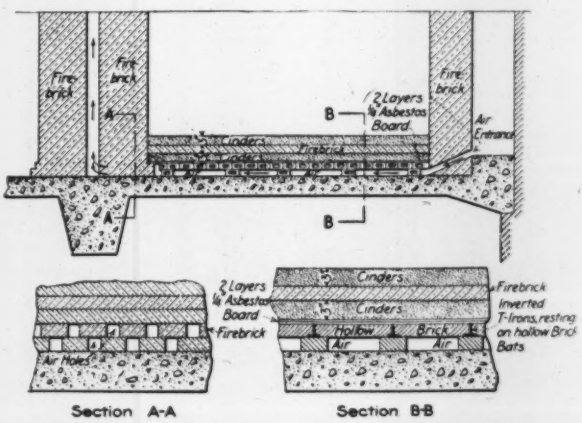
outside. A maximum of 45 yd. per hr. can be handled. An 8-in. spiral riveted asphalt-dipped discharge pipe has been used with very good results in handling this abrasive material.

Each of the pumps has a capacity of 1,050 gal. per min., and the swamp is 1,800 ft. distant. This arrangement was worked out by Anderson Dana, of the Upper Hudson Stone Co., and R. L. Sites, of Brown & Sites Co., 30 Church St., New York City.

Insulating a Concrete Floor Under Boilers*

Two new boilers were installed on a reinforced-concrete floor. After being used for a short time cracks were noticed in the concrete, caused by expansion due to the excessive heat. A little later a brick wall that separated the two boilers had to be taken out because the heat was so intense that the wall started to give way. This

*Reprinted from "Power," Sept. 5, 1916.



DETAILS OF CONCRETE-FLOOR INSULATION

wall was 34 in. thick, with outer layers of firebrick and the interior of common brick, and rested on the concrete floor. The concrete under the wall was found to have begun to dehydrate. On taking up some of the insulation under the combustion chambers, which consisted of a 1-in. layer of 85% magnesia over the concrete, two layers of common brick and a top layer of firebrick, it was found that the magnesia had become pulverized and the concrete dehydrated on the surface.

Asked to suggest some method of overcoming the difficulty, the Ferro Concrete Construction Co., Cincinnati, Ohio, proposed the method shown in the accompanying cross-section. The method depends for its protection largely on the circulating air space between the combustion chamber and the floor, and in the separating wall. Holes were punched through the outer wall to allow the outside air to enter, pass under each boiler setting and up through the slot in the center wall.

This air space under the boiler setting was formed by inverting $2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{5}{16}$-in. steel tees, placed the length of a brick apart, and held up by pieces of hollow brick 2 ft. apart. The tees were covered by a layer of hollow brick, thus forming an air space above the brick. The surface of this brick was flushed with a sealing of clay, which was covered in turn by two layers of ¼-in. asbestos board, section *BB*. Over the board was placed a 3-in. fill of cinders, a layer of firebrick and another 3-in. fill of cinders. Under the brick wall, the air space was formed by placing the two lower layers of brick as shown in section *AA*, leaving air spaces between the bricks. When the wall itself was replaced, it was built of firebrick.

The boilers have now been in operation for some months and no further trouble has been experienced. A vigorous current of outside air is constantly drawn through the air holes. The roof of the air space formed by the hollow brick is pretty hot, but the bottom formed by the concrete has a temperature of about 250 deg., as measured by a pyrometer at various points. This temperature is not high enough to do any damage to the concrete.

Locomotive Crane on Trestle Pulls Coffer-Dam Piles

The piers of the Susquehanna River Bridge of the Philadelphia & Reading Ry., at Milton, Penn., were built in steel sheetpile coffer-dams 20x40 ft. in plan, about 38 ft. deep with about 10 ft. penetration in the sand of the river bottom.

FIG. 1. COMPLETED PIER COFFER-DAM FOR READING BRIDGE AT MILTON

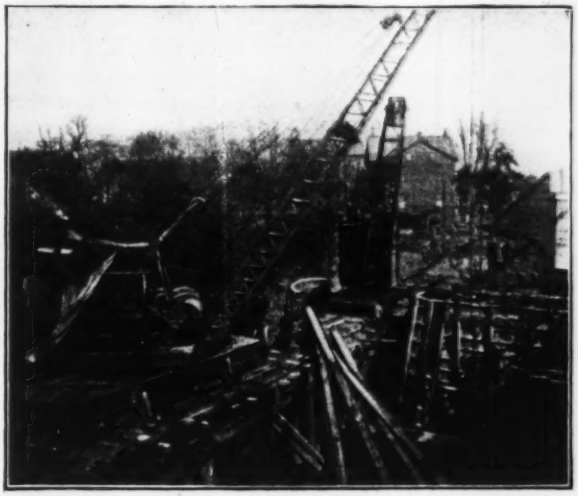


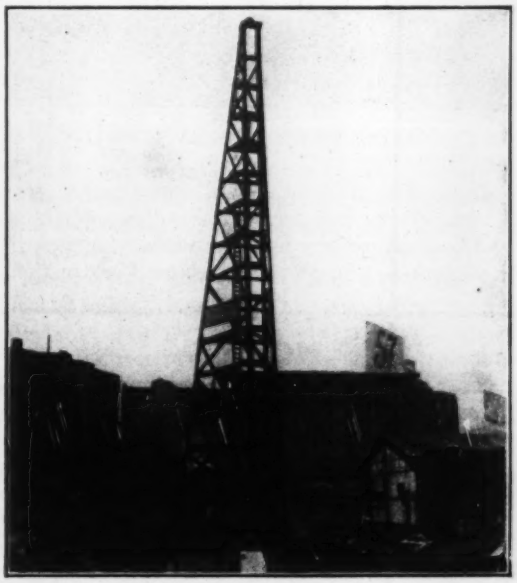
FIG. 2. LOCOMOTIVE CRANE PULLING SHEETPILING, MILTON, PENN.

The concrete was placed from a timber working trestle built along the line of the bridge. When the piers were completed, it was decided to recover the piles by means of a McMyler locomotive crane already on the job. Fig. 2 shows this work in progress. No serious difficulty was experienced, and the piling was recovered at a low cost.

This was an excellent opportunity to use an inverted steam hammer, but the most suitable make of hammer for this service was not on the job. The Robert Grace Contracting Co. built the piers and pulled the coffer-dam sheetpiling. F. P. Kemon was superintendent.

World's Tallest Piledriver for 115-Ft. Piles

The new Southern Pacific general-office building bounded by Market, Steuart and Spear St., San Francisco, requires in the foundation work approximately 2,500 piles 115 ft. long, driven until the heads are 15 ft. below street level. For this work, the Healy-Tibbitts

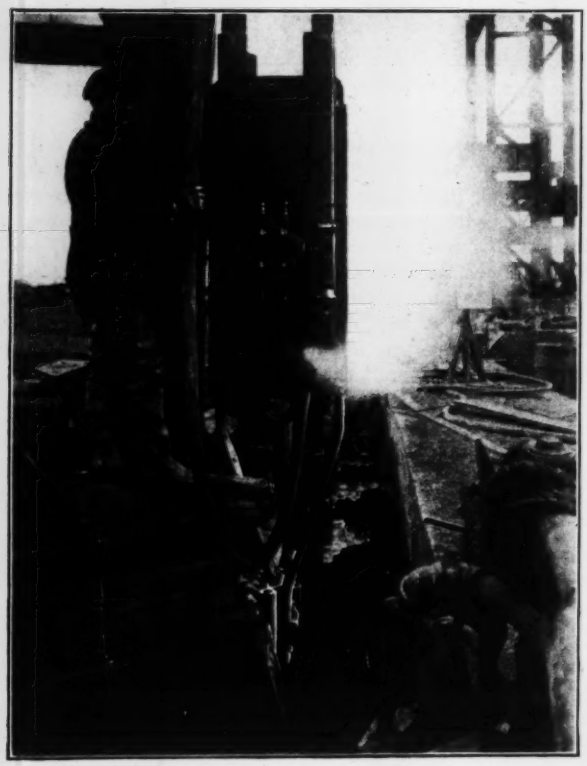


TALLEST PILEDRIVER PUTTING 130-FT. PILES 15 FT. BELOW STREET

Construction Co., contractors for the foundation, have built, and now have in operation, a piledriver believed to be the tallest on record. It is 133 ft. from the bottom of the sill to the top of the headblock. A second driver—a duplicate of the first—is in course of construction, and will be in operation in a few days.

Coffer-Dam Piles Easily Pulled by Inverted Hammers

The construction of the intake and discharge condensing-water tunnels at the new power house for the Buffalo General Electric Co. requires 344 tons of steel sheetpiling in 35-ft. lengths for coffer-dams. This piling



PULLING SHEETPILES AT BUFFALO

was driven and redrawn several times by an inverted McKiernan-Terry hammer. This hammer weighs 5,000 lb. and is shown pulling in the accompanying view. The work is done by the Stone & Webster Construction Co.

NOTES

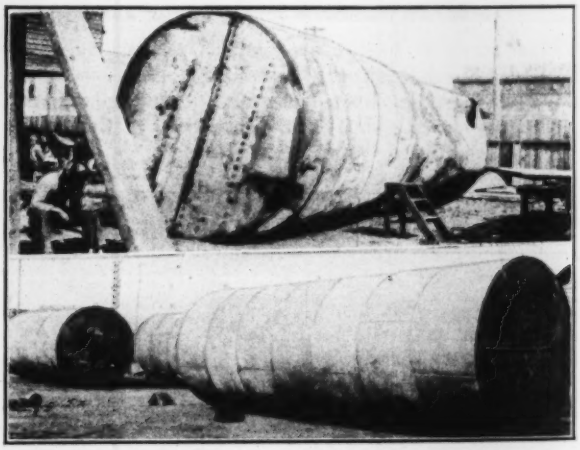
Spot Test for Purity of Creosote Oil—The absorption spot test for the purity of creosote oil is defined by Homer Cloukey, of the Forest Products Laboratory, Madison, Wis., as follows: "Allow six drops of the sample of oil to fall upon the surface of clean, white blotting paper. If tar, carbon or dirt is present it is very easily observed, as it quickly segregates at the center. The paper should be laid away in a flat position for a few hours in a place free from dust. If then examined, foreign matter will be observed in a distinct zone in the center of the spot; the outer zone will very readily indicate the character of the oil." In order to establish some idea of the sensitiveness of the test for tar, carbon or dirt in creosote oil and to make it fairly quantitative, Mr. Cloukey made a series of six tests using carbon-free creosote oil and adding to it in definite graduated amounts carbon in the form of lampblack. The results of the series obtained showed an increasing density of the free carbon ring in the center of the spot as the amount of the lampblack was increased, and further indicated that an amount of carbon so small as 0.005% is easily shown by the test. In percentages greater than 0.5%, however, the amount of foreign matter in an unknown sample would be difficult to determine by comparison. The admixture of tar to creosote oil in actual practice can be roughly estimated by the size and density of the inner zone and the general character of the spot. The heavy tar does not diffuse with the same rapidity as that of the lighter creosote. The absorption spot test has been adopted by the Diesel Engine Users Association of England for the examination of oils used for fuel in this type of engine.—Thomas F. Murphy, Republic Creosoting Co., Indianapolis, Ind.

What Happened to a Brick Pavement without a concrete foundation or adequate underdrainage, when it was subjected to a few days' traffic of a 7-ton motor truck loaded with new paving brick, is shown in the accompanying illustration of a

BRICK-PAVED STREET IN MORGANTOWN, W. VA., UNDER MOTOR-TRUCK TRAFFIC

street in Morgantown, W. Va. Unfortunately the city has many miles of pavement similarly constructed.—C. S. Pratt, Morgantown.

Repairing Damaged Oil Tanks—Twelve oil tanks 30 ft. long by 10 ft. in diameter that had been badly damaged by explosions during a fire were recently repaired by the Johnson & Barry Steel Co., North Birmingham, Ala. The tanks



ONE OF THE DAMAGED TANKS BEFORE AND AFTER REPAIR

were completely cut apart and straightened out, the rivets being removed by "rivet busters," and then the plates were riveted again. Information of this job is furnished by the Chicago Pneumatic Tool Co., whose tools were used.

Editorials

Sewer-Gas Effect on Sewers Needs Investigation

The possible disintegration of concrete in sewer arches under the action of sewer gas is the subject of controversies of infrequent but insistent recurrence. The very great preponderance of concrete sewer structures absolutely unharmed by any such action would make almost negligible the few cases where decomposition has been reported, were it not for the fact that all too frequently emphasis is laid on the actual case rather than on its rarity. There has been a tendency to follow this course in the case of the Chicago stockyard sewer, which is the subject of an article on another page of this issue. Considerable local publicity has been given to the case, and it is probable that the commercial interests in competition with concrete will, with business sagacity, endeavor to bring it to the attention of as many engineers as possible. For that reason we have thought it worth while to present the facts of the case as they appear to a representative of the clay and brick interests and to present at the same time a statement prepared by one of the editors of this journal.

In a consideration of the conditions here shown, engineers should remember that the sewage passing through the stockyard sewer is by no means of an ordinary character. The sewer forms the outfall for the very stale effluent from what is practically a huge septic tank formed by an arm of the Chicago River, into which sewers discharge and which has a sluggish flow to this outfall. This channel is now being filled up and will soon cease to exist, but at present the sewage flowing through it is probably as strong as any in this country. It is far worse than the ordinary discharge in a low-grade sewer where there is sluggish flow. If concrete decomposition were to be expected anywhere, it would be here.

At the same time comparable sewage conditions do undoubtedly exist elsewhere, especially in purification tanks, where no structural deterioration has been noticed. The mere fact, then, that the Chicago sewer is affected is of subsidiary importance. What is important is to find out, first, if the same sewage would in time have a similar effect on concrete or brick of the highest excellence and, second, how the concrete as it exists in the Chicago sewer would be affected by sewage of varying quality. Such an investigation should repay the Chicago authorities.

Meanwhile, lacking these comparable data, the conclusions to be drawn from the Chicago case appear to be practically the same as those which have been drawn from the other reports of disintegration in sewer arches. These are that if given a high enough sulphur content, the sewage will throw off gases which, when mixed with the oxygen of the air and in the presence of moisture, form an acid sufficiently strong to attack concrete of ordinary integrity. Furthermore, this same action will take place not only in the mortar between brick or tile lining, but in brick and tile lining itself under certain conditions. The cases where it does occur, however, are so few that it is fair to assume that with a concrete of sufficient density or one protected by some special coating or waterproofing, or with a brick or tile of sufficient glazing or high enough burning, the probability of disintegration is quite small.

Prompt Unloading of Freight To Prevent Car Shortage

There is good reason to believe that the present fall will witness the most serious car shortage that has ever taken place in the history of American railroading. While the crops of 1916 are not as heavy as in previous years, they command a high price and the surplus will be moved to market as rapidly as the cars can be obtained. In addition to this, business is at a high pitch of activity the country over—the iron and steel mills are working to the limit of their capacity, an enormous volume of manufactured goods from all parts of the country is being carried to the seaboard for export, and congestion in handling freight at tidewater makes more or less delay necessary there.

In years of previous car shortage there has been a tendency to throw the whole blame upon the railway companies for not providing cars enough to meet every possible demand of shippers. At the present time business men, appreciating what the railway companies have recently gone through and what they are now facing, will not be quite as ready as formerly to expect everything from the railways and to forget their own obligations. The fact has been forced home upon the business public that car shortage may be due quite as much to delay of shippers in unloading and loading cars as to delay of the railways in moving them. Warnings have been issued by the Federal Trade Commission, the Interstate Commerce Commission and various state public-utility commissions urging receivers and shippers of freight to hold cars the least possible time in the interest of the public welfare.

To make this warning effective, however, something that will appeal to the individual shipper from the dollars and cents point of view is necessary. The Demurrage Committee of the American Railway Association, after a conference with the National Industrial Traffic League, is seeking authority from the Interstate Commerce Commission to raise demurrage rates to a point that will give receivers of freight a larger incentive to unload cars promptly. It is proposed, after allowing two free days for unloading, to charge $2 for the third day, $3 for the fourth day, $4 for the fifth day and $5 for the sixth and each succeeding day the car is detained.

Another excellent means of utilizing cars to the best advantage is pointed out by the Universal Portland Cement Co. in a statement issued Aug. 30. If purchasers in ordering cement by the carload will make their order read "one car, any size," "two cars, any size," etc., it will permit the cement companies to load up to the limit of

their capacity whatever cars are obtainable from the railways. Where an order is for a fixed number of barrels, however, it is often necessary to send out a car only partly loaded. This practice not only raises the cost of freight on the cement, but utilizes cars inefficiently at a time when every car ought to be rendering its maximum service.

Making Water Consumers Pay for Sewer Service

How many other city departments should a water-works support in addition to its own? Reports from different cities show no unanimity of practice. If the water-works department alone were consulted, there would be general agreement that it should be self-supporting and stop at that. Unfortunately, many water-works departments are compelled to turn their revenues into the general city fund and to accept with as good a grace as possible what the city council appropriates to them for operation and maintenance. Although the city accounts are generally so kept as to yield no certain knowledge of actual financial results, beyond a few crude data, it is common for the city council to grab all the water-works receipts it dares and use them to keep down taxation.

George G. Earl, general superintendent and chief engineer of the Sewerage and Water Board of New Orleans, La., a man well known in the water-works field for his valuable studies of water rates, cheerfully concedes in his annual report for 1915 that sewerage operation and maintenance may rightly be met from water revenue; but he balks at saddling drainage expense also on the water-works, as the city authorities appear to be doing in no small degree. It may be explained that the water-works, sanitary sewers, surface and subsurface drainage of New Orleans are all under the board of which Mr. Earl is engineer.

Mr. Earl's consent to meeting sewerage costs and argument against meeting drainage costs from water revenue are as follows:

> It is logical that water rates should cover the cost of maintenance and operation of both the sewerage and water-works systems. The function of the sewerage system is to carry away the water which has been delivered by the water-works system after it has been used, and in most cases the benefits derived from the sewerage system are actually in proportion to the consumption of water. This argument, however, does not hold as to the drainage operation and maintenance, in which the benefits derived by individuals bear no relation whatever to their consumption of water. The rapid substitution of subsurface drainage and the throwing of the burden of the cleaning and maintaining of such subsurface drains upon the Sewerage and Water Board without a sufficient budget to properly attend to it are responsible for the growing cost of drainage maintenance; and the inability of the city to provide a fund to accumulate for the renewal of machinery and for emergencies necessarily involves an increasing cost of drainage operation. Water rates established only to maintain and operate the water-works system cannot possibly take care of sewerage operation and maintenance in addition and also meet these increasing demands for the maintenance and operation of the drainage system.

Every water-works man will agree with Mr. Earl's plea against supporting drainage works from water receipts; and most will concede that there is considerable force in the argument that sewerage benefits are proportionate to water consumption. Granting all this, we should be surprised if many water-works engineers or superintendents would agree to having water rates cover sewerage as well as water-works operation. Conditions at New Orleans may make this permissible or even desirable. Elsewhere there has been general agreement that there should be no annual charge for sewerage service and that the operating and maintenance charges should be met by general taxation—however the first cost may be distributed.

There is much to be said for having each municipal service stand on its own bottom. Nevertheless, Mr. Earl's reasoning is worthy of thought. If water consumers must bear the burden of other city departments, the sewers may perhaps be loaded onto the water taker's back with more ease and less injustice than any other city service.

Modernizing Specifications in the Navy Department

In times past the requirements in connection with bids for Navy Department supplies have been so numerous and complicated that on the smaller contracts it was often the case that the only bids received were from men who had made a special study of these requirements and who bought goods from regular dealers to fill the contracts. The Government was thus obliged to pay a profit to a middleman, in addition to the regular retail price, to cover the cost of complying with its red tape.

The Chief of the Bureau of Yards and Docks of the Navy Department, Admiral Harris, has recently made some radical departures in that bureau's specifications to bring them into line with general practice in letting contract work elsewhere. These changes are embodied in the specifications for the structural shop at the Norfolk navy yard, for which bids were opened on Sept. 2.

The first notable innovation in the specifications is the abandonment of lump-sum bids and the adoption of unit prices in their place. The specifications contain an estimate of the quantity which will be required under each item, with a statement that the United States reserves the right to increase or decrease the quantities without change of price per unit, provided the variation does not exceed 25%. It is of course evident that this clause decreases the risk to the contractor, since he is certain of receiving payment for the actual amount of work performed. The change is furthermore of great advantage to the engineer, since it enables him to alter the plans for the work as construction proceeds, as very often becomes necessary to secure the best results, without feeling that he is either placing on the contractor undue hardships or giving him unearned benefits.

Another feature which is particularly noteworthy is that the time of completion of the work is not specified. Each bidder is required to give his own estimate of the time in which he will guarantee to finish the work. Liquidated damages are fixed in the specifications for each day that the work is delayed beyond the time named by the bidder. In comparing different bids for completion at different times they are valued on the basis of the time element and penalty.

In the specifications for the new Norfolk shop the liquidated damages are fixed at $250 per day, this being considered as a fair rental charge which the Government will lose by every day the building is unavailable for use after the date when it should have been completed. The contractor may obtain an extension of the allowed time in case of unavoidable delays. These are defined as

delays resulting from causes beyond the control of the contractor, "such as acts of Providence, fortuitous events, inevitable accidents, abnormal conditions of weather or tide, general strikes, or delays caused by acts of Government." On the other hand, delays in securing delivery of materials on account of rejection of materials on inspection, or changes in market conditions, or time taken in submitting, checking and correcting drawings, or inspecting materials are not regarded as unavoidable.

While the specification provides, as above stated, a penalty for noncompletion of the work within the agreed time, it does not provide a bonus in case the work is finished in advance of the agreed time. If the liquidated damages for delay in fulfillment of the contract are made only equal to the fair rental value of the completed work, it seems clear that the contractor should be entitled to a bonus for anticipating the specified date, as well as subjected to a penalty for delay.

Water Refining for Cities

Recent advances in water-purification are summed up in the two words "water refining" by Prof. William T. Sedgwick in an address published in the July number of the "Journal" of the New England Water-Works Association. Water refining includes disinfection ("final sanitation"), clarification, toxination (of algæ by copper sulphate), decolorization, deodorization, softening and deferrization.

Aside from disinfection, all these methods of water treatment are aimed to meet the demands of "taste and comfort" rather than health. It signifies great progress that a people who a decade or two ago could hardly be coaxed or driven into providing money necessary to save themselves from typhoid fever should now be demanding disinfection to make assurance doubly sure and insisting on many refinements besides.

It is well to bear in mind that disinfection is a matter of water refining rather than a primary agent of purification. Rightly used, it is extra insurance against either a light and occasional possible pollution or else against temporary aberrations in a filtration plant—besides, of course, catching and killing the few straggling germs that invariably escape filtration. When pollution is heavy and constant, recourse should be had to filtration; disinfection will still be in demand as a refinement.

Because there has been so much advance in water-refining methods and because so many cities have adopted one or more of them, it should not be thought that little remains to be done in the water-purification field. Hundreds of cities have not yet adopted even the crudest of the primary processes, and thousands have yet to enter upon water refining. Practically all surface supplies must ultimately be filtered, and there are few even now that should not be at least subjected to disinfection—a remarkably cheap and highly effective form of insurance. Coagulation and sedimentation are found to be extended, and the beneficent effects of long-time storage of surface supplies will also gain in practical recognition. Softening and iron removal have only made a start in America and are bound to progress rapidly. The removal of manganese, a refinement not mentioned by Professor Sedgwick, may or may not be needed often; but probably the very few plants for the purpose now built or under construction will be increased many fold.

The 22nd New York Engineers Make Good in the Field

Constructing Camp Whitman—the mobilization point of the National Guard of New York—was perhaps the most extensive operation ever performed by a militia engineer organization, and it is proper that the excellent work of the 22nd New York Engineers, now at McAllen, Tex., be not only described but commended. Many New York engineers will recall the surprise of the public at Governor Glynn's order, of Dec. 24, 1914, disbanding the 22nd Regiment. According to reports only 15 of the old officers were retained and something like 25% of the enlisted men were discharged. The daily press of New York City was rife with caustic allusions to tango soldiery. That these allusions were not without justification is proved by Major-General O'Ryan's statement which he made at the time. He is quoted as saying:

> The 22nd Regiment is steeped in old infantry methods. All the officers and men capable of performing the services expected of engineers will be assigned for duty after the regiment is disbanded and the battalions reorganized.

All officers retained in the unit had first to win the approval of Lieut.-Col. Lucas, Chief Engineer Officer of the New York militia, who as president of a special examining board passed upon the qualifications of the officers. Colonel Lucas reorganized the two battalions and thereafter commanded them. During his régime 18 new officers have been appointed, the officers of the corps now numbering 37.

Quebec Bridge Suspended Span Collapses

Just as this page of *Engineering News* goes to press on Sept. 11 a member of its editorial staff sends the following dispatch from Quebec:

> Center span collapsed and fell in river at 10:50 a.m. Cantilevers intact. No structural evidence left. Only circumstantial evidence available as to cause.

The engineering world was amazed when the south cantilever of this great bridge fell nine years ago; but words are inadequate to express its sensations at the news that again this great bridge enterprise has suffered an unprecedented disaster. It is true that the process by which this huge 5,000-ton span was to be hoisted off the scows and lifted bodily nearly 150 ft. in the air was unprecedented in the annals of engineering. It is also true, as those who followed the description of the process in *Engineering News* of Aug. 31 can understand, that every step of the process had been carefully planned in advance, and to all appearance its success was as fully assured as is possible with any piece of engineering work, even work carried out as ordinary routine.

On later pages of this issue will be found a description of the span which collapsed and as full details as are available of the disaster. Engineers may well suspend judgment on this most deplorable event until full information can be given.

This latest failure can only delay for a short time the completion of the great bridge. The suspended span is after all of moderate proportions compared with the gigantic cantilevers which it will connect; and in no long time the successful placing of a new suspended span will be recorded.

Letters to the Editor

Venturi Meters on Lively Lines

Sir—Mr. Hazen's article, "Venturi Meters Inaccurate on Lively Lines," in *Engineering News*, Aug. 17, 1916, is very interesting, and his illustration and explanation of the theory showing that over-registration would be, under the conditions of his example, as high as 41%, appear very plausible; and this is the theory that has been held for a long time by many of those experienced in this class of work. The writer, however, has never been perfectly satisfied with that view, for the reason that venturi-meter registers are designed to measure the flows directly by integrating the square root of the head, and these integrations are in the nature of a differential proposition, practically instantaneous. The only reason why Mr. Hazen's theory would be effective is that there is a certain lag between the movement of the mercury and the movement of the register. Nevertheless, his statement is correct that under rapidly fluctuating flows the result is an over-registration.

There are two practical methods of correcting this. One is by the use of well-charged air chambers, and the other is by means of a compensating device consisting in attaching to the two small venturi-tube and register-connecting pipes a U-tube of very large dimensions in comparison therewith and in which is kept a heavy liquid, whose difference of surface level will fluctuate very little compared with the fluctuation of difference in pressure in the two small pipes, the quieting effect being due to the inertia of the heavy liquid. The writer views the difficulty as one of inertia, and it has its analogy in the current meter and from the fact that in a turbulent river a raft will float downstream faster than the average velocity of the surface of the water. The explanation in the case of the current meter is that velocities greater than the average will be impressed on the moving vane; and if there were no friction, the vane would revolve at the speed corresponding to the maximum velocity. While this difficulty may be overcome by a certain form of construction of the current meter, it is certain that for the ordinary type those that are light in construction give more accurate results in a turbulent stream.

Mr. Hazen infers that makers of venturi meters may have obtained their coefficient of tubes by tests with reciprocating pumps. This is quite unlikely. Some of the principal firms manufacturing this type of instrument have made very exhaustive tests by means of gravity flows through venturi tubes of various sizes and ratios, the water being weighed in the most accurate manner in well-equipped hydraulic laboratories, and the venturi heads being obtained by means of water columns, using air, tetrachloride of carbon, and directly by means of hook gages. In many cases, all these methods were used simultaneously, while at the same time the results of weighing were checked by measuring the discharge from the venturi tube through a triangular-notch weir. The results, to the writer's personal knowledge of at least one of these firms, are unquestionably accurate. They show that there is a variation in the coefficient depending on the ratio of the venturi tube. In some cases the coefficients are as high as 99%; in others as low as 95%. If we were going to take a fair average we would use about 97½%. But the coefficient varies slightly at velocities above 4 ft. per sec., through the throat and quite materially below this, the coefficient being less at the low velocities, and the variations corresponding to a rather typical curve.

J. W. LEDOUX,
Chief Engineer, American Pipe and Construction Co.
Philadelphia, Aug. 18, 1916.

Sir—Mr. Hazen's note on this subject in *Engineering News*, Aug. 17, 1916, interests the writer, as he had some experience of a similar nature during the development of the apparatus for integrating the flow through a Deacon waste-water meter. An integrating meter of this type, known as the Kelvin bulk meter, is now manufactured and sold by the Palatine Engineering Co., of Liverpool. The natural movements of the pencil in a waste-water meter are very lively, and a short trial was sufficient to show that it was impossible with any degree of accuracy to integrate the corrected movements direct with a clock-driven time disk, as the under-registration or slip was so considerable. The lively movements were subsequently reduced to as small an amount as desirable by the interposition of a cataract, and no further trouble was experienced.

The important point, however, to notice in these waste-water meter experiments is that the error in registration was in the opposite direction to that referred to by Mr. Hazen and was proved to be due to another cause which is of general interest, as it must also affect any case of integration with a time disk where the friction roller of the registration counter is impressed with lively lateral movements.

An experimental apparatus was prepared in which the friction counter wheel was mounted upon a vibrating mechanism with adjustments so that both the speed and the frequency of the lateral vibrations and the weight on the counter wheel could be varied. This apparatus was put upon a clock-driven time disk and its true registration determined without the lateral movements. These figures were compared with the subsequent registration when lateral movements of various amplitudes and frequencies were impressed upon the counter. It was found that the slip increased both with the frequency and the amplitude of the lateral movements of the counter to such an extent that it might be inferred that with very high vibrations no registration whatever would take place. The weight upon the friction wheel did not appear to have much to do with the amount of slip. The movements produced somewhat the same effect as smoothing down and lubricating the time disk. A somewhat parallel case occurs when opening or closing a telescope tube, where it is found that by a slight rotating movement in addition to the direct effort the tube can be much more easily moved longitudinally.

It would appear that both the over-registration referred to by Mr. Hazen and the under-registration referred to here may be made negligible by checking the lateral movements of the registration counter; and this should be very easy with the venturi meter, as it is only necessary to put partly closed cocks or orifices in the communication pipes between the venturi tube and the float chamber in the same way as is frequently done with pressure gages on lively lines.

W. GORE, Consulting Engineer,
John ver Mehr Engineering Co., Ltd.

Toronto, Ont., Aug. 29, 1916.

Dangerous Timber Roofs Due to Incompetence

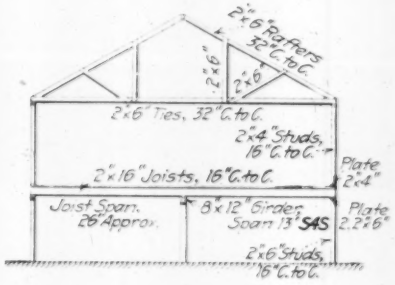
Sir—The editorial in *Engineering News*, July 13, 1916, with regard to the failure of the Atlanta, Ga., skating rink, and that at Salt Lake City, is to the point. While, owing to the lack of plans of the buildings, it may be impossible to determine the initial cause of the failures, yet the lack of longitudinal truss and roof bracing is so apparent in the Atlanta building as to warrant suspicion as to responsibility for the failure. Unfortunately, lack of bracing is too common a fault of many structures of this type, and results largely from failure on the part of the owner to investigate the capability of his architect, coupled with a desire to get the cheapest building possible. This statement does not, of course, excuse the lack of proper inspection of the plans by the building departments.

Were sufficient information available, it would be interesting to know the stresses in the truss members. With the very flat curve of the truss, the chord stresses must have been of considerable amount, even with the small weight of roof supported. Laminated chords, especially when but sparsely spiked, as these are described to be, are not efficient as columns. The spiking is subject to a large initial stress in holding the boards to the constrained position as arcs of the curve of the truss, and a small addition of load in the chords must have increased the stress in the spikes sufficiently to allow them to give slightly and the individual boards to slip past each other. Thereafter the strength of the chord would be the sum of the strengths of the boards acting as separate columns. These small columns would also have to resist bending in addition to axial stress, due to the eccentricity of the thrust or pull on the mid-section between panel points. The photographs show the chords bent to a more or less sinuous curve without fracture.

Lack of a tie at the floor line is also a criminal omission, and the fact that the ties provided were an afterthought indicates that neither the architect nor the inspector who passed the plans (if they were ever examined by the building department) had any conception of arch action.

The writer wishes to call attention to another feature of the trusses. As far as can be judged from the photographs, none of the web members are dapped into the chords, and apparently the only way in which the webs could transmit their components of stress into the chords was by means of whatever friction existed plus the resistance of possible toenailing. Possibly the architect believed, as the writer has found not a few designers (including an engineer or two) to hold, that the web stresses in a timber truss go chasing themselves down one diagonal and up another in a vain attempt to find ingress to the chords, until they come to the end of the truss, when they give up the chase in despair, and conclude to remain in the webs.

The writer had occasion recently to examine a dance hall, concerning which complaints had been made in regard to its swaying while dancing was in progress. The hall proper was about 52x80 ft., the dance floor being the second floor. The typical construction of the building is shown by the accompanying sketch. The construction indicated illustrates the common way of framing a wooden building on this coast; that is, constructing a stud wall, cutting and capping it, laying the floor joists on the caps, placing the plates of the second story on the joists, framing the second-story studs, capping and placing the ceiling joists and roof rafters on the caps. Ordinarily the cross-partitions, diagonal wall sheathing, etc., furnish the necessary stiffness to the building. In this case there were no cross-partitions,



CROSS-SECTION OF FRAMING OF DANCE HALL; UNSAFE

and even the end partitions were of doubtful rigidity, due to openings.

The roof, with a span of 52 ft., was carried by partially trussed 2x6-in. rafters and ceiling joists, 32 in. on centers, the trussing in the center portion being omitted, with the result, of course, that all the trussing was ineffective. For good measure, apparently, the architect had built real trusses, of two 2x8-in. chords, 2x8-in. compression webs, and steel tension rods. These trusses were 10 ft. on centers, approximately. Unfortunately, as the "trussed" rafters ran the same way as the real trusses, the latter were prevented from accomplishing any results in the line of supporting the roof, despite their latent usefulness. It is hardly necessary to state that there was not a vestige of bracing between the walls and roof, or for that matter in any portion of the building with the exception of two kneebraces underneath the second floor, and in the line of the second-floor girders, or in the stiffest way of the building.

The second-floor joists were found good for a live-load of 87 lb. per sq.ft., and the second-floor girders for 36 lb. per sq.ft., both figures being at 1,800 lb. per sq.in. bending stress. The result of this construction was that the roof between end walls had settled very perceptibly, and the somewhat ornate plaster ceiling was quite badly cracked. Not only did the floor vibrate very unpleasantly, but the movement in the walls was very perceptible while dancing was in progress.

Engineering News is rendering a real service in giving publicity to failures such as the Salt Lake and Atlanta collapses, and the accounts of these should be brought to the attention of the public at large, to the end that any structure for public use be made safe beyond the shadow of a doubt. H. D. DEWELL.

58 Sutter St., San Francisco, Calif., July 19, 1916.

Unit Prices for Street Mains in the Buffalo Gas Case

Sir—Through personal knowledge of the facts involved, I have been particularly interested in the article in your issue of Aug. 24, "The Determination of Unit Prices in Appraisals for Rate Making," by Judson C. Dickerman.

Permit me, from a knowledge of the facts and in defense of the engineering profession, to comment on Mr. Dickerman's statement, as based on the opinion given by the Public Service Commission, 2nd District, State of New York, in the Buffalo Gas case.

The opinion was written by F. W. Stevens, after he had retired from the commission of which he had been chairman. Mr. Stevens' opinion of the estimates of numerous engineers as to unit prices, Mr. Dickerman says, ends up with the clear and forceful statement that fixing a unit price for an extensive system of gas mains from data submitted by engineers or inferred from contractors' bids on other jobs is nothing but a guess, and the commission declined to make the guess.

Mr. Dickerman's last paragraph, as his article is condensed in your paper, is as follows:

> The valuations as of date of appraisals are based on unit costs which are so much the result of the judgment of the engineers making them that they are little better than opinions. And the opinion of no one man or associated group of men, particularly where such group or individual is liable to be influenced directly or indirectly by the interests employing him, can be taken with safety as to a fair and reasonable value.

Unfortunately, the case as presented by Mr. Dickerman is plausible. But what are the facts? To commence with, Mr. Stevens' opinion not only ignores facts but contains statements as to the testimony which, to speak in guarded language, are misleading.

I have before me a complete analysis which I made of the opinion at the time it was first placed in my hands. This analysis shows that Mr. Stevens made many comparatively unimportant errors in quoting facts and figures from the testimony. Even these minor errors serve to show, however, that Mr. Stevens' statements need to be carefully checked before acceptance.

But let me pass to more serious errors. I wish particularly to refer to the evidence as to unit costs of street mains, as that is the point emphasized in Mr. Dickerman's misleading article. On p. 47, Mr. Stevens gave a table of unit prices for street mains which he says were testified to by a gas company's witness in the previous 1907 case, and then he compares that table with the unit prices testified to by the gas company's witness in the case now under consideration. On this comparison and the failure to agree between the two statements, both of which he ascribes to the gas company's witness, he bases most of his ridiculous criticisms of engineers' testimony.

At first we of the gas company were puzzled to discover the source of the 1907 unit prices, as quoted by Mr. Stevens and falsely attributed to the gas company. We were unable to find any warrant for these unit prices in our testimony, but finally we found that these unit prices were those testified to in 1907 by the late W. D. Marks, *a witness for the city*. In passing, I may state that Mr. Marks' unit prices failed to contain certain necessary costs of laying, which fact we demonstrated in our cross-examination of this inexperienced witness.

The error here introduced by Mr. Stevens is carried through page after page of his subsequent arguments against the company. Time after time he refers to this Marks' testimony as the testimony of the gas company. There is no misunderstanding, therefore, upon which to base an apology for Mr. Stevens.

I now come to a part of Mr. Stevens' opinion which I do not care to characterize. Mr. Stevens attempts to show that the unit prices testified to by the company were simply opinions, that these opinions varied, and so "the commission declined to make a guess." On page 68, Mr. Stevens says:

> It (the commission) has no evidence as to the general character and conditions of the soil to be encountered in making excavations; . . . there is nothing showing what the cost of labor would be . . . no engineer sworn before us knows anything about the conditions to be encountered of soil or obstruction in the streets.

On page 100—"General Conclusions":

> 3. There is no means of ascertaining the actual present conditions of the street mains which constitute the larger part of such property if such condition is material in determining what if any depreciation should be deducted from the cost of reproduction new.

Without quoting further from this remarkable opinion, it may be claimed that Mr. Dickerman was justified in assuming that the unit prices for street mains were based upon opinion and not upon any evidence of fact. That is, Mr. Dickerman was so justified in adopting the views he expresses, based upon this one opinion, if he feels that the "opinion" of a commissioner can be accepted on its face value and without examination of the evidence itself.

The fact is that the evidence submitted on the condition of the mains was unusual for its completeness. I offered to have the streets opened wherever the city or the commission so desired, provided the representatives of the commission, the city and the gas company were present together when all the examinations were made.

While these witnesses were testifying, Mr. Stevens had in his hands the report of the commission's engineer who took full part in the examination. Mr. Stevens afterward acknowledged to me personally that the paper he was constantly referring to was this report and that his report agreed practically with the testimony of the gas company's witness. He also expressed his dissent at the methods employed by the city's witness above referred to.

Samples of pipe taken out of the streets were produced in court by the gas company, and a few samples which were too bulky to be so examined were examined by Mr. Stevens at the gas works, Mr. Stevens and I walking together to the gas works from the courtroom.

If there ever was testimony offered in a rate case which was based upon the facts laid bare to all parties, it was this testimony of the Buffalo Gas Co. on street mains. Furthermore, the gas company's testimony was not opinion testimony, but it was supported by the estimates constantly being made in my office in connection with the extensions authorized, and later checked by the construction costs. What better testimony could be given, especially when presented by honest men of the widest experience as operators? If this is not valid testimony, there

is no such thing, and engineers' estimates of cost are valueless indeed.

In conclusion, let me say that I am not particularly interested at this late day in this question on behalf of the Buffalo Gas Co., but I am concerned for fear the estimates of capable engineers may be discredited.

ALEXANDER C. HUMPHREYS,
President, Buffalo Gas Co.

165 Broadway, New York City, Aug. 26, 1916.

Full-Load Secondaries Were Eliminated at Quebec

Sir—In an editorial on the work at Quebec in your number of Aug. 17, there is a slight error.

The bridge is so designed that the trusses have the intended geometrical shape—with straight chords and straight, continuous web members—when the main span is fully loaded and the anchor arms are unloaded. For this loading the main members of the cantilever arms have all practically their maximum stresses. With this loading the only secondary stresses from vertical loads in the main members of the cantilever arms are those from pin friction, especially at the large pins at the main shoes. These are provided for by additional section in the bottom chords near those pins. When the main span is unloaded, there will be considerable secondary stresses in the bottom chords of the cantilever arms, but these are of no importance since they do not coexist with maximum primary stresses.

The bottom chords and the inclined end posts of the anchor arms also have their maximum stresses for the above loading, and they also are so set as to become straight under that loading. Their only secondary stresses from vertical load are those due to pin friction, and these are alone of much importance near the main shoes, where they are provided for by additional section.

There arise also secondary stresses from lateral deformation by wind pressure. These are also considered where they are of sufficient importance to call for increase of section under the provisions of the specification, when added to the other secondary stresses. This specification allows larger unit stresses for the sum of all the stresses than for the sum of the primary stresses. Secondary stresses are therefore only considered where they are a large percentage of the primary stresses.

A very important kind of secondary stress, especially in compression members, arises from differences of temperature of different parts of the same member, when some are exposed to the sun, some in the shade. A temperature difference of 25° F. was assumed, and the large secondary stresses arising therefrom were considered.

More in detail, the elimination of secondaries due to vertical load was accomplished as follows: The lengths of all the main members of the cantilever system were so chosen in the shop that with the main span loaded with 10,000 lb. per lin.ft. of bridge and the anchor arms unloaded, these members have what we call their geometrical lengths, or the lengths which would be the proper shop lengths if there were no deformations. With this loading all the panel points of the chords and those of the web members continuous over two panels will be in straight lines.

This loading produces nearly maximum primary stresses in all the main members of the cantilever arms and in the chords and inclined end posts of the anchor arms and suspended span. With it the secondary stresses in these members are very small. What secondary stresses there arise are due to the moments of the pin friction at pin connections. The pins are lubricated and the pin friction coefficient is less than 2% at the start, but will gradually increase. The friction coefficients were ascertained by tests at McGill University. The members will adjust themselves to such mean positions on the pins that the opposing friction moments balance.

It was assumed that the coefficient of pin friction will increase until the tangential force at the pins produced by the friction is 10% of the axial force. At most of the pins turning will come to a stop under the increasing friction, but at the main shoes it will continue under extreme loads.

Only those secondary stresses are of importance which increase the sum of all coëxisting stresses in a member. Secondary stresses which occur with loadings giving small primary stresses are not of importance. When the bridge is unloaded there will be large secondary stresses in the bottom chords and end posts of the anchor arms and in all the main members of the cantilever arms, but these do not coëxist with maximum primary stresses and they are smaller than the maximum moving-load stresses in these members.

When the main span is fully loaded, the anchor arms unloaded, there are no secondary stresses in the anchor arm except from pin friction and other than moving and dead loads. When the anchor arm is fully loaded and the main span unloaded there are secondary stresses in the anchor arm, but as regards the chords and inclined end posts they are of no importance because the primary stresses are small. In the web members the secondary stresses coëxist with maximum primary stresses, but, as these members are slender, their secondary stresses (with a few exceptions near the main shoes) are not of sufficient size to demand an increase of section.

When the bridge is fully loaded there are secondary stresses of considerable size in the inclined posts of the anchor arms which join the main shoes. The corresponding bending moments go to the shoes, not to the cantilever arm. The secondary stresses in the top chords and in the tension members going to the top of the vertical main post from the anchor arm are very small. They produce small bending moments in the top chords and tension members of the cantilever arms, which are negligible.

The essential principle is to reduce those secondary stresses which coëxist with maximum primary stresses. This can not be done where, as in stiff laterals, opposite and equal stresses occur.

The wind stresses and those from differences of temperature are of opposite signs, and the resulting secondary stresses occur together with maximum primary stresses and must be considered.

A slight inaccuracy results from the fact that the load assumed in calculating the deformations of the members was 10,000 lb. per ft. of bridge while the load used in calculating the primary stresses is 5,000 lb. per ft. of track followed or preceded or followed and preceded by two of Cooper's E60 engines.

JOSEPH MAYER,
Principal Assistant Engineer, Board of Engineers,
Montreal, Aug. 21, 1916. Quebec Bridge.

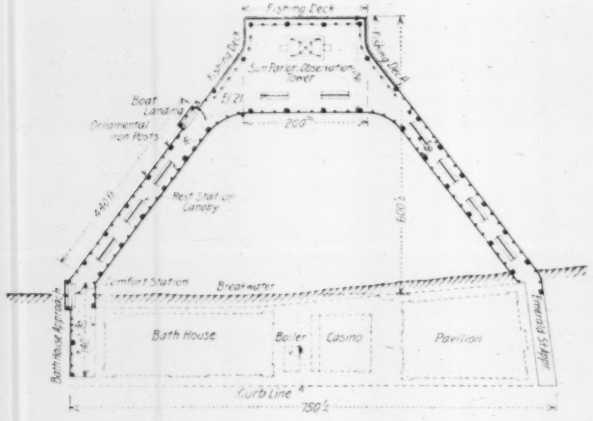
Concrete Ocean Pier Built in Form of Hollow Triangle

By C. L. Roberts*

A reinforced-concrete pleasure pier recently completed at Redondo Beach, Calif., is peculiar in that in plan it consists of two separate piers extending from the shore diagonally toward each other and meeting at the ocean end in a platform or cross-pier. The site is at a submarine cañon extending from several miles off shore to within about 500 ft. of low-water mark, forming a wide navigable channel to deep water. The beach to the north and south of this cañon runs to sea on a gradual gradient of 6%, but the cañon bottom has approximately a 1:1 slope to indeterminate depth. Years ago a wooden pier was built from shore out along the side of the cañon to accommodate shipping, but in 1913 heavy seas wrecked the structure. Bonds were thereafter voted in the sum of $125,000 to replace the old pier with a modern reinforced-concrete municipal pier.

This structure, shown in Fig. 1, extends out to sea about 600 ft. As shown in the drawing, the out-to-out distance along the shore between the legs is about 700 ft. Each leg is 36 ft. wide, and the outer platform is about 200x160. The combined pier and shore frontage permit a continuous promenade of about three-quarters of a mile in length.

*Superintendent of Construction, MacArthur Brothers Co., Redondo Beach, Calif.



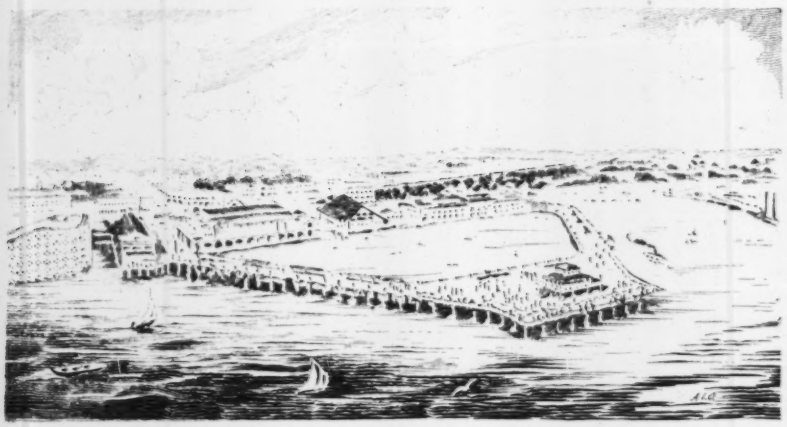


FIG. 1. PLAN AND SKETCH OF NEW MUNICIPAL PIER AT REDONDO BEACH, CALIF.

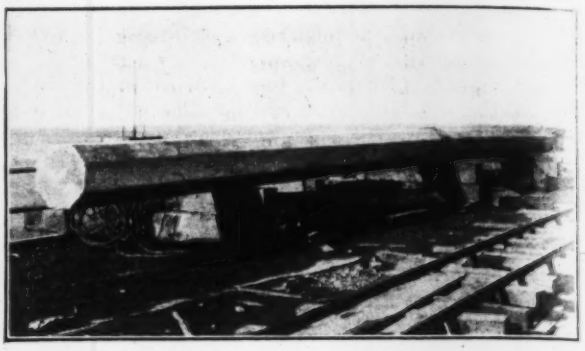


FIG. 2. ONE OF THE CONCRETE PILES, REDONDO BEACH PIER

The structural design is a straight beam-and-girder type; that is, bents made up of three transverse reinforced-concrete piles tied together by longitudinal tie-beams carry a transverse floor-beam and joist system on which the reinforced-concrete floor rests. The piles, Fig. 2, varying in length from 20 to 85 ft. and in diameter from 20 in. (reinforced with 5/8-in. bars) to 24 in. (reinforced with 1 1/8-in. bars), with a spiral hooping of 4-in. pitch, were precast and driven by jetting. The tie-beams are of 12x24-in. section reinforced with four 1 1/2-in. rods, while the transverse floor-beams are 16x24 in. in section, reinforced with four 5/8-in. square twisted rods. Running into these floor-beams are 4x10-in. joists reinforced with one straight and one bent 5/8-in. bar. In the outer wharf all the girders are 16x24 in. in section.

On each side of the approaches as well as on the legs of the pier the structure has cantilever beams which carry an overhang in the floor. These beams are 16x42 in. in section, with reinforcement of two 1-in. bars at the top and two 3/4-in. bars at the bottom, with the necessary shear stirrups. On the southerly approach as well as on the southerly leg a comfort station and a fish market are provided. These structures are carried on 14-ft. cantilevers duly designed for the extra load. Around all four sides of the outer wall a 10-ft. cantilever has been constructed to serve as fishing platforms. The increased depth of the transverse and longitudinal beams at the outer pile line acts not only to provide for the increased moment in shear, but as a haunch to take care of indeterminate stresses thrown into the structure on account of the heavy seas which predominate in March. The whole pier was designed for a live-load of 100 lb. per sq.ft. plus the dead-load, and the city reserved the right to place a test load of 200 lb. per sq.ft. upon any selected panel. Owing to the fact that in actual construction the entire wharf area had to be used for material loading and carrying, the test was omitted. The whole structure contains approximately 3,500 cu.yd. of concrete and 300 tons of reinforcing steel. A new method had to be devised in building the pier because the frontage of the work is the main-line right-of-way of two railways where constant travel did not permit any permanent organization for the casting of piles.

handling concrete materials, or storage. In consequence the first lot of piles were cast on the small areas available and were transported to the work and driven. On the part of the pier thus constructed, platforms were erected on which the subsequent pile casting and concrete mixing were carried out.

The piledriving equipment consisted of two heavy piledrivers of the skid type, one 36 ft. high and the other 50 ft. high, with a Worthington heavy-duty pump used for the jetting. The pump was placed on a platform midway between two legs of the pier and connected up with the power plant of the bath-house, water being piped through 6-in. pipe out to and onto the two legs of the pier, an additional pipe being laid as the work progressed. Pressure was maintained at about 100 lb. at the pump and was entirely adequate until penetration reached 20 ft.; then it became difficult to overcome the friction with this pressure. Jet lines were run from the main pipe lines to the driver by means of two 2½-in. fire-hose, then to the tops of the gins of the drivers by two 2-in. pipes and then to the jets.

The jets were handled by having a pair of lines leading from the head lines of the drivers and to the drums of the engine By keeping the engine on the driver running constantly the jet man could plunge the jets up and down in much the same way as the well driller works his drills. The jets themselves were bushed down to a 1-in. nozzle and acted like a hydraulic giant.

Falsework for carrying the piledrivers out over the permanent work consisted of 12x12-in. caps 40 ft. long on bents of five piles each. These bents were driven on 10-ft. centers and so placed that the concrete bents came between the wooden-pile bents. This prevented any tendency on the part of the jet to sluice out any of the wooden piles. Great care had to be exercised in order to avoid such an occurrence.

The wooden-pile bents were sway-braced with 4x12-in. timbers, and longitudinal girts of the same size timbers were run out. For the transportation of the piling

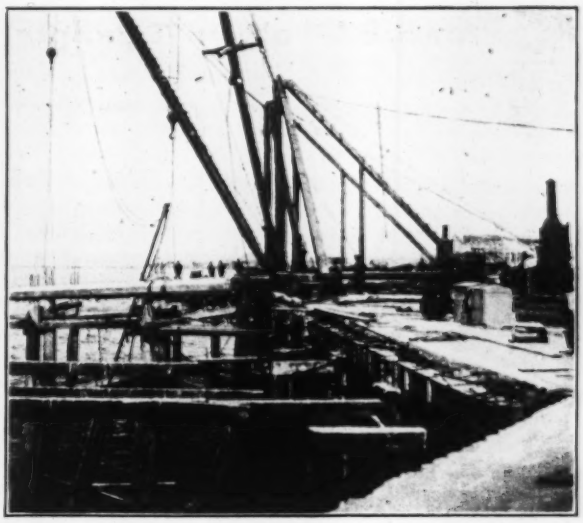


FIG. 4. SWINGING ONE OF THE CONCRETE PILES INTO PLACE

12x12-in. timbers were used for railroad-track stringers. These timbers were taken out as the piledriving progressed and utilized for forms to hold up the concrete floor. As soon as a wharf area was constructed, a pile platform was laid and piling cast on the completed work. The concrete equipment included two small mixers which permitted constant moving out onto the work as it progressed. The material was handled to these mixers by means of small 24-in. gage tracks, and typical station-man cars were shoved out by hand, alternating sand and rock with cement.

This system worked splendidly until deep water was encountered. Then it became difficult to move the drivers, the larger of which weighed 31 tons. From then on, the smaller driver drove the bulk of the falsework. When deep water was reached, there were times when

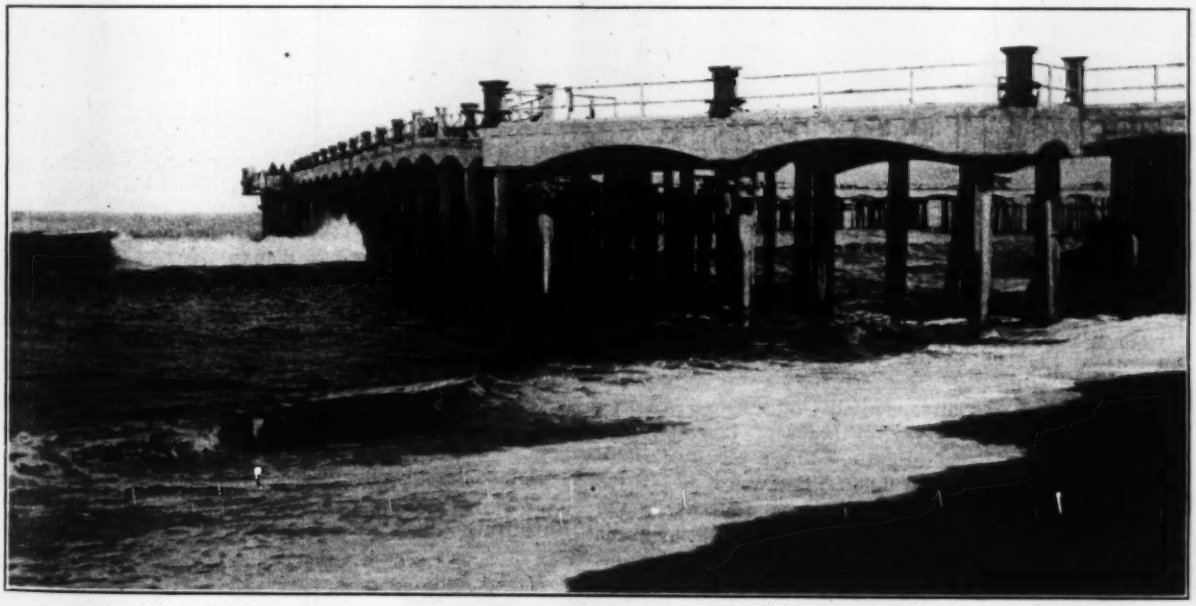


FIG. 3. VIEW OF ONE ALMOST COMPLETED END OF THE REDONDO BEACH PIER

the rigs stood 60 ft. above the floor of the ocean with no means of holding the falsework from pulling out from beneath the drivers. Escapes from a complete launching of the rigs were narrowly averted. On Mar. 22, 1916, one of the heavy northers swept in without any warning. Within two hours of the first blow the entire outer works were scoured and the two drivers lost. Great difficulty was experienced in getting the equipment out.

Toward the end of the pier, owing to the trouble of supporting the piledrivers and excessive lengths of piling, derricks had to be resorted to in driving the piles. These derricks were worked in pairs, the mast being placed on 20-ft. centers. The stiff-legs were lashed to the piles beneath the dock. Concrete piles were rolled to the derricks, and four slings were placed upon the longer lengths. Two derricks would then swing their loads to sea and lower the entire load to the bottom. One derrick having the foot of the pile would let go and swing out to help the derrick having hold of the top; the two would then up-end the pile. Collars were placed upon the concrete piles, and the men steering the jets would ride the pile until it got down to its proper depth.

The work was designed by George W. Harding, of Los Angeles. MacArthur Brothers Co., of New York, Chicago and San Francisco, was the contractor, with the author as superintendent and C. S. Robinson as office manager. J. R. Kirby represented the engineer.

New Knowledge on Standard Reservoir Mouthpiece

By W. S. Pardoe*

Recent experiments conducted in the hydraulic laboratory of the civil-engineering department of the University of Pennsylvania are of value in explaining the action of the water within a standard short tube and also in correcting many indefinite and erroneous statements contained in standard textbooks on hydraulics.

The tube used in the experiments was 2 in. in diameter, 6 in. long, tapped for ⅛-in. pipe at 12 points spaced in an irregular helix. Each connection was controlled by means of a ⅛-in. cock. A common header was connected to the vacuum gages, water being used for the lower negative pressures and mercury for the higher ones. The general arrangement of apparatus is shown in Figs. 1 and 2.

Thirteen experiments were made with heads varying from 2.5 ft. to 47.5 ft. An examination of the results (Fig. 3) shows that the negative pressure h is about 0.860 of the head H; that it is confined to the inner half of the tube for heads up to 37.5 ft. when the vacuum is 31 ft., or very close to atmospheric pressure. It will also be noticed that up to this head the negative pressure varied, indicating the presence of water at a negative pressure and not a vacuity, as usually represented in works on hydraulics. The curves for heads of 42.5 and 47.5 ft. indicate by their general flatness that a vacuity existed. Furthermore, h is no longer 0.86 H, but approaches 34 ft., or a perfect vacuum. This condition is reflected in the falling off at the coefficient of discharge C_D. At or about 52.5 ft. head the tube ceased to act as a short tube, but discharged a stream similar to that of a standard orifice, this change occurring when the expanding jet ceased to fill the end of the tube. Upon lowering the head the tube did not necessarily again discharge as a short tube, but occasionally did.

The action of the water and consequent losses of head can be explained in a theoretical way by assuming the loss before the *vena contracta* to be similar to that of a standard orifice and the loss after this point to be due to sudden enlargement. The average results of tests on a

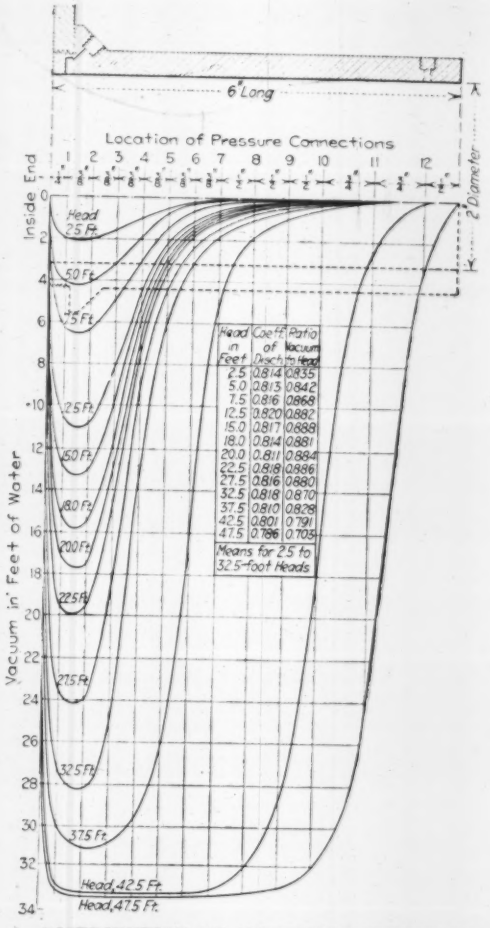


FIG. 1. DISTRIBUTION OF NEGATIVE PRESSURES IN 2-IN. STANDARD SHORT TUBE

2-in. standard orifice gave the following results: Coefficient of discharge, $C_d = 0.5955$; coefficient of contraction, $C_c = 0.602$; coefficient of velocity, $C_v = 0.989$.

Let V be the velocity in the full section of the tube and v the velocity in the *vena contracta*. Then

$$H = \left(\frac{1}{C_v^2} - 1\right)\frac{v^2}{2g} + \frac{(v - V)^2}{2g} + \frac{V^2}{2g}$$

$$= \frac{V^2}{2g}\left[\left(\frac{1}{C_v^2} - 1\right)\frac{1}{C_c^2} + \left(\frac{1}{C_c} - 1\right)^2 + 1\right]$$

Substituting the foregoing values,

$$H = 1.498\,\frac{V^2}{2g}, \text{ or } V = 0.816\sqrt{2gH}, \text{ or } C_D = 0.816$$

which check accurately the average coefficient for heads

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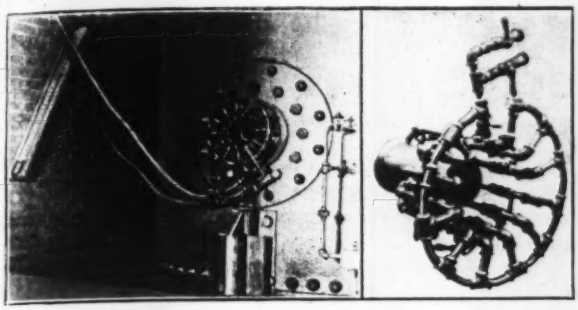


FIG. 2. EXPERIMENTAL SHORT-TUBE MOUTHPIECE

up to 37.5 ft. The negative pressure h can be computed as follows:

$$h = H - \left(\frac{1}{C_v^2} - 1\right)\frac{v^2}{2g} - \frac{v^2}{2g} = H - \frac{V^2}{C_v^2 C_c^2 2g}$$

$$= H\left[1 - \frac{C_D^2}{C_v^2 C_c^2}\right]$$

Substituting values, the negative pressure $h = -0.875\ H$, which again checks fairly closely with the experimental result obtained.

The foregoing theoretical analysis depends entirely on the assumption that the jet within the tube contracts much the same as a jet from a standard orifice and then reëxpands and fills the end of the tube, the surrounding medium being water in a more or less stationary collar up to 37.5 ft. head, and being a vacuity for greater heads. To verify this a glass tube 2 in. in diameter and 12 in. long was substituted for the tapped brass tube, and the following observations were made:

1. Up to a head of 18.5 ft. the tube flowed full with no appearance of air within it. Hence if contraction does take place, the jet must pass through a surrounding collar of water in a more or less stationary or eddying state.

2. At 18.5 ft. head a slight amount of air was seen to disengage from the water at the inner end, extending for about $\frac{1}{2}$ in. into the tube. This condition progressed until the head was 28.5 ft., when the inner end of the tube was surrounded by a dense white fog extending about 1 in. into the tube.

3. At 38.5 ft. head the appearance was that of snow or loose foam extending about 6 in. from the inner end of the tube. At 44 ft. head the disturbed condition extended to the outside end of the tube, the inner end appearing as a clear-cut jet showing very distinctly the contraction and reëxpansion on which the foregoing theory depends.

4. At 56 ft. head the jet ceased to reëxpand sufficiently to fill the end of the tube, and it discharged as a standard orifice. The appearance of the glass tube discharging under various heads is shown in Fig. 3. Notice the diverging character of the stream under a head of 42.5 ft.

The coefficient of discharge for heads greater than 37.5 ft. may be computed by considering the inside end acting as an orifice discharging under a head of $H + 34$ ft., thus:

$$Q = C_D a \sqrt{2g47.5} = C_d a \sqrt{2g\,[47.5 + 34]}$$

or

$$C_D = C_d \frac{\sqrt{81.5}}{\sqrt{47.5}} = 0.781$$

Which again approximately checks the result obtained for a head of 47.5 ft. This would indicate that although the contraction and reëxpansion are not apparent for heads of less than 37.5 ft., it nevertheless occurs and the theoretical method of analyzing the subject is correct.

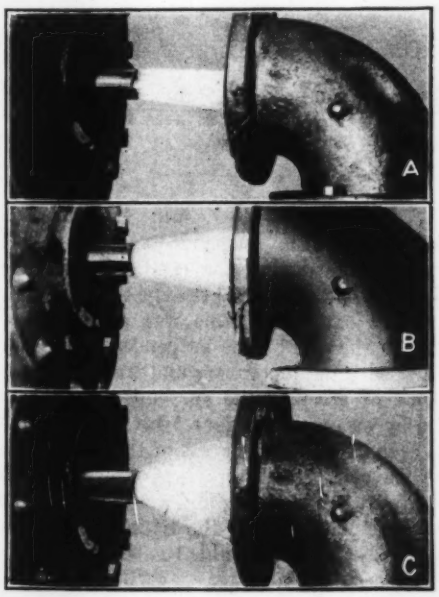


FIG. 3. TYPICAL DISCHARGES FROM GLASS SHORT TUBE
A—Under 2.5 ft. head. B—Under 20 ft. C—Under 42.5 ft.

Rules for Applying Federal-Aid Road-Improvement Act

Under date of Sept. 1, 1916, the United States Department of Agriculture has issued a booklet of rules and regulations to be applied in the expenditure of federal-aid road money.

The act is interpreted to apply to those states where the actual expenditures are made and the work is done wholly by the counties, but supervised in a general way by a state highway department. No federal money will be paid for work which can not be proved to have been economically done. Bids for the work must have been advertised, and the contract must have been awarded to the lowest responsible bidder, unless it be satisfactorily proved that it was advantageous to accept the higher bid; otherwise the federal aid will be prorated to the lowest bid and not the actual cost of the road.

The Office of Public Roads must be notified in advance of a contract letting, so that a representative may attend. No part of the federal money is to be used directly or indirectly for the payment of a premium or royalty on any patented or proprietary material, specification, process or type of construction, unless purchased or obtained by open competitive bidding at the same or less cost than unpatented articles or methods equally suited to the same purpose.

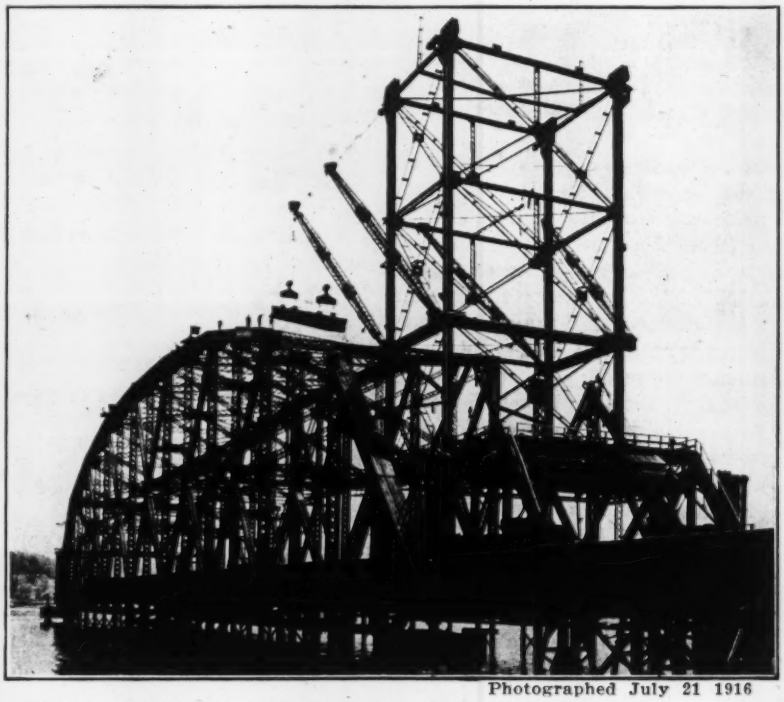
The rules and regulations governing surveys, plans, specifications, estimates, contracts, construction work, costkeeping and payments are so comprehensive that the United States Office of Public Roads and Rural Engineering will have a dominating influence in all work for which application for federal aid is made.

Erection of Quebec Bridge Suspended Span

By A. J. Meyers*

The only large step in the construction of the Quebec Bridge that has not been described for the information of the engineering profession is the erection of the 640-ft. suspended span on shore previous to floating to the bridge site and hoisting into place.[1] The importance of the structure warrants completing the record, as is attempted in the accompanying photographs.

The suspended span is 640 ft. long center to center of end supports and 88 ft. wide center to center of trusses. The trusses of the span have the top chords curved to a parabola, the depth at the hip being 70 ft. and at the middle of the span 110 ft. center to center of chords. The web is a subpanel Pratt system with main verticals, compression posts, and main diagonals, tension members. The main panels vary in length from 65 ft. in the end panels to 80 ft. for the panels at the center of the span. For the bottom chords throughout and for the first main tension diagonals of the web, eye-bars were used. All the other truss members were of built-up construction. The top chords were pin-connected at all the main panel points with shop or field splices at the intermediate subpanel points. The web members were connected and riveted together at the main panel points of the top and bottom chords by means of gusset plates which engage the pins of the top and bottom chord members.



Photographed July 21 1916

FIG. 1. COMPLETING ASSEMBLY OF QUEBEC SUSPENDED SPAN AT SILLERY COVE

All truss members were manufactured, shipped and handled in subpanel lengths. Nickel steel was used throughout for all the main truss members and the bottom lateral system, but the top lateral system and the sway bracing were built of carbon steel, as well as the minor web members which carried no moving-load stresses. The greatest area of the top-chord members was 434 sq.in., and in one panel of the bottom-chord eye-bars 311.5 sq.in. The top and bottom lateral systems, as well as the sway bracing, were double intersecting systems, designed to take both tension and compression in each member.

The erection site for the suspended span was located about three miles below the bridge site, in the shallow waters of Sillery Cove, where the bed of the river is exposed at low tide. The excavations for the foundations of the falsework bents supporting the trusses and approach tracks were mostly in rock—red shale and sillery. This excavation work was carried on at every low tide occurring in the daytime. On this account and because of the fact that considerable blasting had to be done, progress was slow, three hours' work once and sometimes twice a day being all that could be accomplished. However, this work and the laying of the concrete for the foundations were all completed during the season of 1915. During the fall and winter of 1915-16 the inside staging that had supported the bridge floor and the traveler tracks during the erection of the south anchor arm was taken down and sent back to the contractor's shops at Rockfield, P. Q., to be remodeled into the falsework bents that supported the suspended-span trusses and the approach tracks during the erection of the span. The traveler that had completed the erection of the north-shore anchor and cantilever arms was also taken down and shipped to the site of the suspended span, where it was reërected with the top trusses and traveling cranes left off, the work of erecting the suspended span being accomplished by means of the four 70-ft. 30-ton booms, placed one at each of the four corners of the traveler.

The traveler and its bracing were figured to resist a 30-lb. wind when standing and a 5-lb. wind when moving. No effect of wind during the lifting operations was considered, as no work could be done during winds of appreciable strength.

The dead-load of the traveler was carried on the front and rear truck springs when moving. Under working

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[1]For notes on the erection of the cantilever arms see "Engineering News," Aug. 17, 1916; for progress in 1915, see Sept. 2, 1916; camps and yards, Mar. 5, 1914, and Oct. 14, 1915; anchor arm, Jan. 7, 1915; erection traveler, Mar. 4, 1915; general design, Apr. 30, 1914; special shopwork, May 14, 1914.

conditions these springs were locked, and shims were driven to a snug fit between the front legs of the traveler and the pedestals resting on floor-beams. Reactions from lifting the top chords, which were the heaviest pieces handled in the span, were on the assumption that the members for one truss only would be lifted at one time. If similar members for both trusses were lifted at the same time and placed in pairs, the reactions on the front legs of the traveler were equal, but the reaction on either leg was less than the maximum leg reaction if the pieces were lifted and placed separately. No anchorage, when lifting loads at either the front or the rear of the traveler, was required, except when placing the floor-beams. It was then necessary to outhaul in all cases when the floor-beams were placed more than 72 ft. in advance of the front legs of the traveler; and when placing floor-beams more than 50 ft. in advance of the traveler, the rear derricks had to take a lift on the floor-beams to the rear of the traveler approximately equal to the load on the front derricks. This procedure was necessary in order to avoid overstraining the longitudinal bracing of the traveler.

As shown in Fig. 2, the span was supported during erection on staging bents placed under each panel point. These staging bents were made up of two columns, spaced 6 ft. apart and tied together by means of batten plates and lacing angles. In designing the columns it was assumed that the total weight of the bridge material, the weight of the traveler, including the member being erected, and the vertical reaction from wind on the traveler and span were carried by the outside columns of the bents. The inside columns were assumed to take

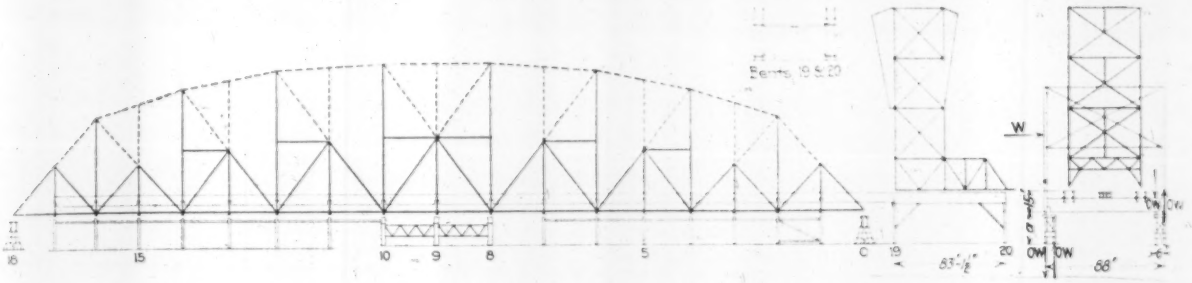


FIG. 2. ERECTION DIAGRAM OF QUEBEC BRIDGE SUSPENDED SPAN AND TRAVELER

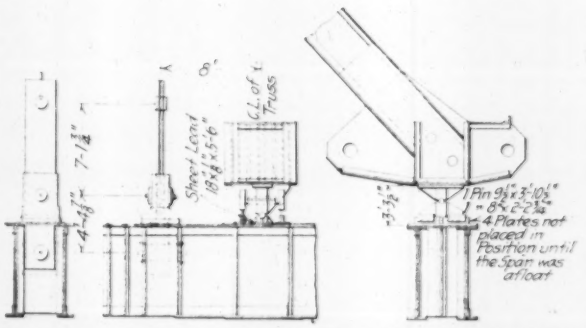


FIG. 4. END SUPPORTS AND LIFTING GIRDER, QUEBEC SUSPENDED SPAN

care of the overturning effect from transverse wind applied at the top of the bent. Bracing placed in the plane of the outside columns took care of the longitudinal wind load on the trusses and traveler, the staging being braced in two sections. The wind load between panel points 1 and 7 was taken care of by the bracing in panel 1-2, and that between panel points 8 and 17 by bracing in panels 8, 9 and 10. The traveler was erected on bents 19 and 20 immediately adjacent to the staging of the span. While standing at bent 19, the traveler erected staging bents 1 and 2, the longitudinal bracing between bents 1 and 2 and the bridge members in panel 0-1. The traveler then moved forward, erecting staging and longitudinal bracing, floor-beams, bottom chords, bottom laterals and web members, except the upper half of the main diagonals and the vertical substruts, as it advanced, until it reached bent 17. The traveler then moved backward and completed the erection of the trusses by placing the top chords and upper web members as well as the top laterals and sway bracing. In Fig. 2 the members erected as the traveler advanced are shown in full lines. Dotted lines indicate the members placed on the return trip.

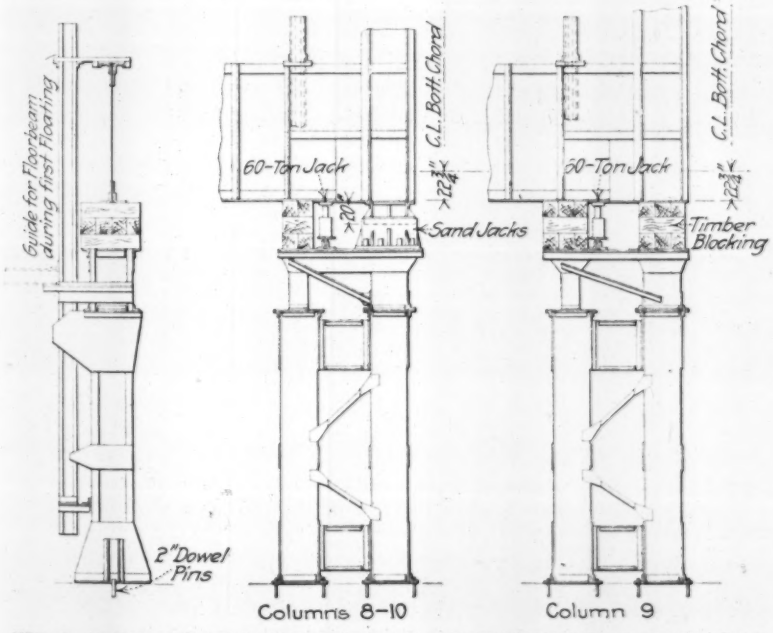


FIG. 3. STAGING BENTS AND JACKS FOR SUSPENDED-SPAN ERECTION

As illustrated in Fig. 3, sand jacks were used at the even panel points, directly under and with tops bolted to the vertical posts of the trusses, to transfer the load to the outer columns of the staging. Timber blocking was used for the same purpose at the subpanel points and also between the floor-beams and inside columns of the staging at all panel points. The vertical posts and vertical hangers into which the floor-beams framed had their inside webs slotted to allow the end web plates of the floor-beams to pass through. In order to make this connection the floor-beams were placed on timber blocking over the inside staging columns, about 3 in. above their camber position; and after placing the vertical posts or hangers and bolting the floor-beam connection, the floor-beams and verticals were together lowered to their correct camber positions to a bearing on the sand jacks or timber blocking over the outside staging columns by means of hydraulic jacks.

In the shop the truss members were made of such a length center to center of panel points that after the span was erected and carrying its full live- and dead-load the trusses would have their geometrical shape; that is, the truss members would be straight between main-panel points. On this account the faced ends of the top chords, where these members were spliced in the field, would not come to a square bearing during erection. In order to obtain a square bearing at these points before riveting on the splice material the camber elevations of the lower-chord panel points, during the erection of the span, were calculated so that the top-chord bearing surfaces for the pins at the main top-chord panel points and the bearing faces of the top-chord members at the intermediate panel-point splices would come together just sufficiently to allow the splice material to be temporarily bolted in place with 50% of the field holes filled with bolts.

The span being completely assembled, the timber blocking at the intermediate staging supports and under the floor-beams was taken out, and the sand jacks were lowered until the span rested on its end supports at L0 and L18. The top-chord members then came to a bearing on the pins in the half-pin holes at the main panel points; and the bearing surfaces at the field splices came together, the nuts on the bolts attaching the splice material having previously been loosened. The field rivets in these splices were then driven. All the vertical subposts supporting the top chord members at the subpanel points were made of such a length that these chord members would be straight between main-panel points under the dead weight of the span as it rested on its end supports.

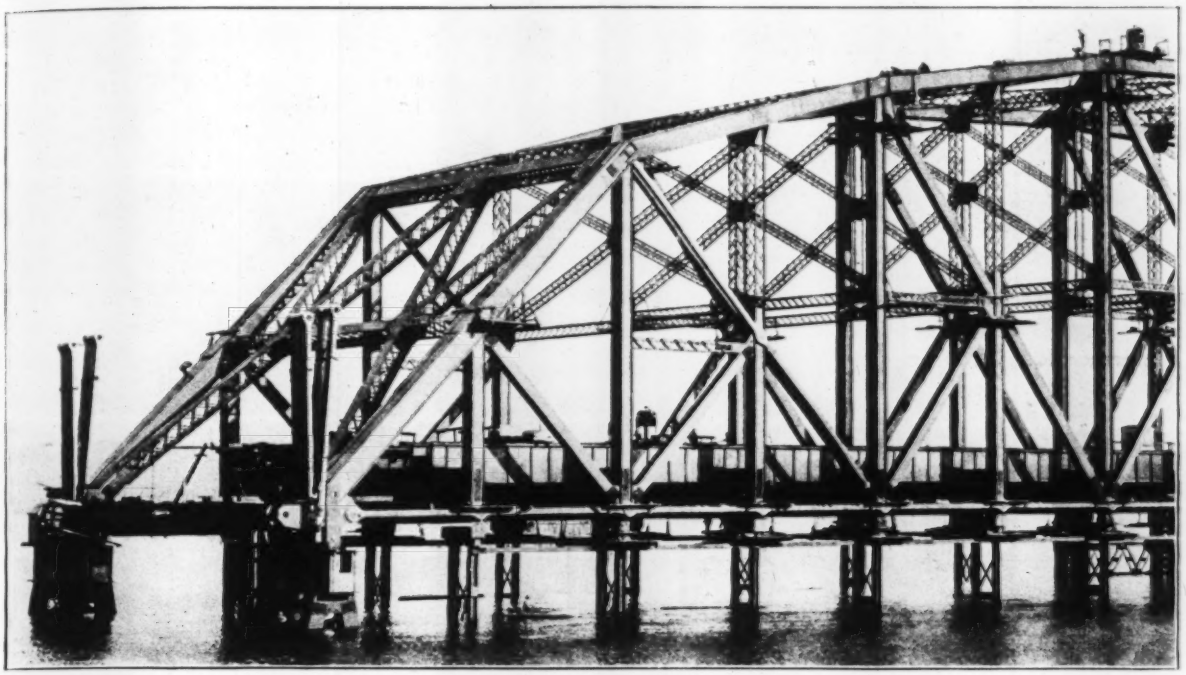
In designing the trusses the main diagonals in the center panels L8-L10, where the web system is double intersecting and statically indeterminate, were calculated to carry only live-load stresses in the lower halves of these members. The upper portions then took the panel dead-load concentrated at panel point 9. In order to insure this condition after the span was erected, the holes in the gusset plates connecting these members at the bottom-chord panel points L8 and L10 had been drilled in the shop to $\frac{15}{16}$ in. in diameter and the holes in the member to $\frac{11}{16}$ in. in diameter. When the members were first erected, these connections were fully bolted with $\frac{7}{8}$-in. bolts; and when the span was swung on its end supports at the panel points L0 and L18, if the panel points L8 and L10 were not at the same distance from a line joining L0 and L18, the lower of the two points was lifted by means of hydraulic jacks, the $\frac{7}{8}$-in. bolts being loosened in both connections. The shims under the higher point were removed as the load was applied to the lower point; and when the panel points L8 and L10 were brought into their correct position with relation to the line joining L0 and L18, they were firmly shimmed against the staging bents. All $\frac{7}{8}$-in. bolts being removed, the $\frac{11}{16}$-in. holes in the members were reamed to $\frac{15}{16}$ in. in diameter through the holes in the gusset plates and the connection fully riveted up before removing the shims.

At the end supporting points L0 and L18 provision had to be made to take care of expansion and contraction due to changes of temperature and the stretch of the bottom chord under dead-load stress after the points of intermediate support were removed. As illustrated in Fig. 4, the span was supported at these points by a series of castings and pins. The arrangement of castings and pins shown was demanded by the conditions of hoisting to place in the bridge. However, to take care of this expansion and contraction, the lower longitudinal pin and its bearing castings were polished and lubricated to insure that the friction would not exceed 10% of the vertical end reaction of the trusses as the surfaces slipped under the end thrusts of the span; and the end bents at L0 and L18 in addition to taking care of the longitudinal wind on the span were designed to resist this horizontal friction force, which was estimated at 300,000 lb.

Quebec Bridge Suspended Span Falls Into River in Hoisting

(Telegram from Harry Barker, Associate Editor, "Engineering News.")

Quebec, Sept. 12, 1916.—The great center span of the Quebec Bridge fell into the St. Lawrence River at 10:50 a.m. Monday, Sept. 11, after it had been hung to the cantilever arms by the erection hanger chains and had been hoisted successfully 10 or 15 ft. From the best information now available, it is believed that six lives were lost in the disaster and six men were injured. I had closely watched the operations of floating the suspended span into position, mooring it to the guide trusses and beginning the work of hoisting, and I saw it collapse. There was no lack of expert eye-witnesses. In fact it is doubtful whether there was ever brought together on this continent a larger assemblage of eminent structural engineers than had gathered at Quebec to witness the final task in the completion of the greatest bridge in the world. The Canadian Society of Civil Engineers had arranged a special excursion of its members to witness the event. No one had any thought of danger in connection with the erection work, but the engineers of the St. Lawrence Bridge Co. decided that it was better for



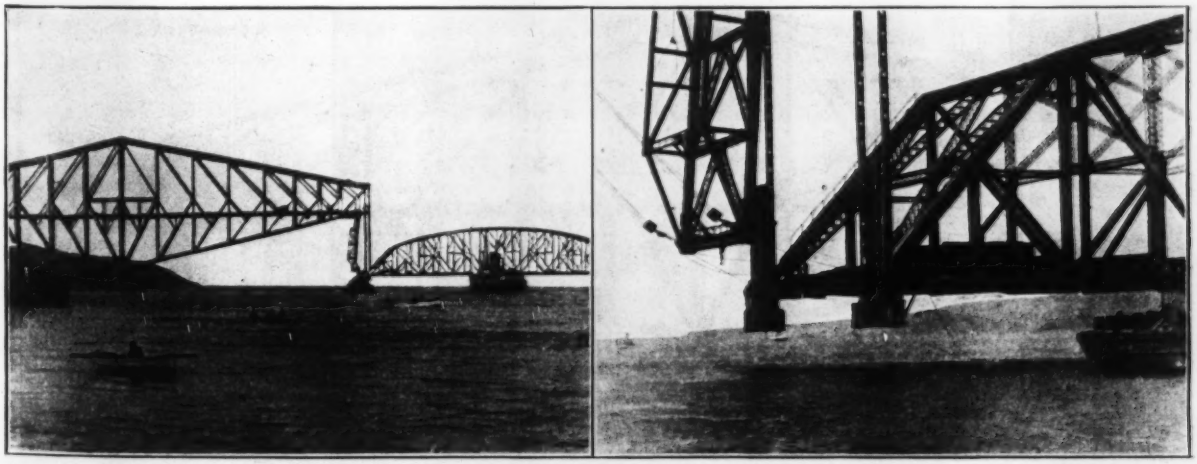
THE SUSPENDED SPAN ON FALSEWORK AT SILLERY COVE
Permanent hanger eye-bars projecting up from end pins; hoisting links projecting up from lifting girders

those in charge of the operating machinery not to be distracted by the presence of any number of guests on the moving span. The visitors, therefore, observed the work from the shore or from vessels in the river or from the cantilever arms above.

The delicate work of transferring the suspended span from the falsework on which it was erected at Sillery Cove, to the scows on which it was to be floated to the bridge site was done during the night. The valves in the bottom of the scows under the span were closed about 1 a.m. As the tide rose, the load was gradually transferred from the falsework supports to the scows and at 3:40 the span was afloat. Further work was then postponed until daylight.

At 4:40 a.m. tugs began moving the span out. At 5:15 the span had been swung across the stream and the shore lines were cast loose. At 5:20 the three-mile trip upstream began, two tugs ahead pulling and five tugs downstream dragging backward to keep the floating mass under control, with an eighth tug, a large seagoing boat, standing by. Aided by the flowing tide, the trip was made in 1 hr. 10 min. At 6:35 the span was in position under the cantilever arms; 7 min. later the auxiliary mooring lines from the steam hoist on the floating span had been run out to the south cantilever mooring truss. Then one after another the four main mooring lines from the electric hoists on the south cantilever were attached. The same procedure was completed at the north end a few minutes later. The whole process of mooring required 1 hr. 5 min.

At 7:25 the downstream tugs eased off and nosed in close to the scows. About the same time the upstream



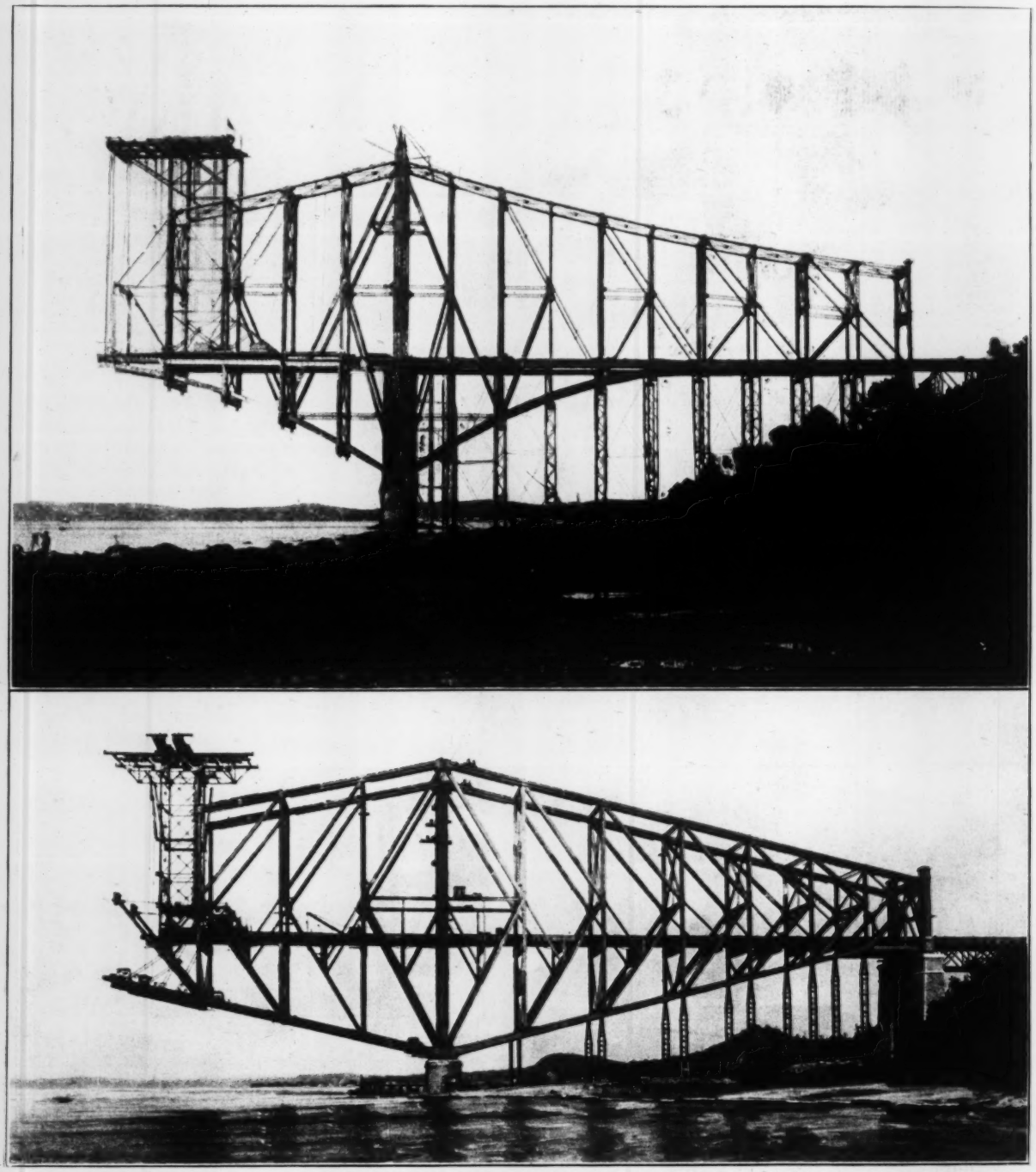
THE SUSPENDED SPAN HANGING FROM THE HOISTING CHAINS
In the detail view, note the short transverse girder, the "lifting girder," under the span corner; the hoisting chains are connected to the short links seen in the view at top of page

tugs also eased off to stand by. Twenty minutes later all lines had been made fast at the north and south ends of the truss and all the tugs had cast off.

At 7:48 the north hanger chains swung forward from back of the mooring truss. Three or four minutes later the chains on the south side were swung down. All four pairs of hangers were connected to the span simultaneously, within a few seconds.

At 8:23 a.m. the erection link pins were seen to be at the bottom of the slot on both north and south hanger chains. As the tide fell, the weight of the span began to be transferred to these chains. At 8:50 the jacks at the north end of the span began working, and practically at the same time the jacks at the south end. At 9:22 a.m. the span cleared the scows, the south end lifting apparently a few seconds first. At 9:25 a.m. the scows on the north side were cut loose from the span. The south group followed in a few minutes.

The scows then drew to one side revealing the spectacle of the great 640-ft. span hanging suspended from



A COMPARISON OF OUTLINES—THE OLD QUEBEC BRIDGE CANTILEVER AND THE NEW, IN PROCESS OF ERECTION—1907 AND 1916

SOUTH HALF OF THE OLD QUEBEC BRIDGE
When completed nearly to center of suspended span as a cantilever structure, the entire half bridge fell

the lifting chains and tumultuous chorus of cheers resounded from the crowds aboard the vessels which crowded the river downstream, and among the many thousands of spectators on both banks of the river. Everybody believed, of course, that the crucial and difficult stage of the work was past and the rest was merely a routine operation. Four lifts of 2 ft. were taken with the north jacks and five lifts with the south jacks. Work was then suspended for a short time to relieve the men before starting on the long and taxing routine of hoisting.

Hoisting Is Resumed After Recess

The lunch interval lasted an hour; then all hands went back to the bridge, and jacking was resumed. The operations worked perfectly as before. The span rose steadily.

At 10:46 a.m. one additional lift had been taken at each cantilever since the lunch recess, making five lifts at the north end and six at the south. At this time on each side of the river the top jacking pins were out; the bottom jacking pins were in, and the top girder was descending for another cycle. The weight of the suspended span was hanging from the lower or fixed jacking girders and everything apparently was stable and secure.

At this moment, something, somewhere, gave way on or about the suspended span, far below the level of most of the observers. What that initial failure was, every engineer in every branch of the profession will earnestly desire to learn, for concededly this bridge represents the height of constructional effort. It is possible here to give only an account of the fall of the suspended span as seen by the editor of *Engineering News* and as recounted by other eye witnesses afterward. These descriptions permit an answer to the puzzle—in fact several answers, wherein lies doubt and confusion for the present. Several combinations of circumstances can be formulated to agree more or less closely with all the evidence at hand.

The Great Span Falls Into the River

At 10:50 a.m. the attention of the closest observers was attracted by sounds that seemed to tell of serious structural failure. To some there was a noise like the discharge of a cannon, almost immediately followed by the lighter cracking of accessory failures, and then slipping and roaring noises, as the span went into the water. It is hard to separate the subjective from what was actually observed in the few seconds while the amazed observers watched the span disappear.

Observers who were clear of the bridge downstream, off the south cantilever, seem to agree that the south upstream corner of the span first slipped off the lifting girder. They think also that at this stage that corner of the span dropped almost to the water, thirty feet below, before the south downstream corner left its lifting girder. To southerly observers, the overturning motion started by the falling of the upstream end continued as an upstream overturning, and simultaneously the end of the upstream truss appeared to fall down as the structure corkscrewed into the water. It appeared further that a considerable part of the span—possibly a third of its length—was submerged before the north ends appeared to slip off their lifting girders. It was not possible to tell from this distance whether one corner at the north end lifted off in the fall.

An important difference is noted in the observation made by a prominent engineer who watched the span from the very tip end of the north cantilever. He declares that the upstream truss at the north as well as the south corner slipped off before either of the downstream truss ends left its seat.

It seemed to observers on the south cantilever as though the great north cantilever must be pulled down by the violence of the shock. Some small disturbance of the cantilever end was apparent from across the river, but the gigantic frame stood the test nobly, and in a few seconds came the realization that it was safe. In the meantime the hanger chains, with the lifting girders on their ends still intact and in place, swung about violently, crushing the wooden inside operating platforms.

On the jacks and platforms of each arm were about twenty men. Only one of these was lost and three hurt, though all the rest had narrow escapes. The chief inspector of the Board of Engineers, Herbert McMillan, posted on the downstream platform, north cantilever, had a leg broken. One man on the upstream platform, north cantilever, dropped overboard and was lost. On the south cantilever one man on the downstream pin platform jumped and with three others was rescued.

Many prominent engineers from the United States and Canada were on the suspended span when the lifting operations began. At the intermission in the jacking operations they came ashore. That saved their lives. The total death roll is still uncertain, but probably only 11 were lost and 6 injured in the great accident.

Both cantilever arms of the structure were thronged with many prominent engineers who had come to see the crowning achievement of the Quebec bridge's history. Besides these visitors, there were engineers and inspectors of the company, and of the Board of Engineers, and many government officials. These men find their sensations, as they were thrown down and about by the oscillations of the cantilever ends set up by the shock of the load release, difficult indeed to describe. Out of the chaos of stories, however, there comes some indication that the downstream cantilever end first went up and returned, while the upstream cantilever end was pulled down and outward. This first motion might have been followed by a series of oscillations of 15 to 18 in.

maximum. It appears clear that each of the K-trusses acted independently, thanks to the flexibility of the lateral connections. Yet there appears to have been no damage to the lateral system for observation indicates no permanent distress of the lateral members. Through the interior of the bridge the longitudinal edges appear still perfectly parallel.

THE CONDITION OF THE FOUR PAIRS OF HANGER CHAINS

The hanger chains, as previously stated, are still there and substantially intact, with the lifting girders hanging at their lower end. *But they are not uninjured or unmarked*, and this of course is significant. Careful note has been taken of their condition in a joint examination by the Board of Engineers and the St. Lawrence Bridge Co.

South Side Hangers—The downstream hangers show the lifting girder at the bottom, the jacking girders at bridge floor, and the top supporting girder, inclined downward in the upstream direction at an angle of 10° or so; the hangers and girders rocked on the top pin. The lower end of the inner hanger, i.e., the one nearer the center line of the bridge, is bent upstream. The lifting girder hung to the chain is battered, the lower or longitudinal bearing pin has disappeared, and the upper or transverse pin has turned and dropped into the place of the lower pin.

The upstream hanger chain shows all girders substantially level transversely. The hangers themselves are not damaged, except that the inner one is scored where it sheared through the end bottom strut of the span. There is evidence of rotation of the lifting girder from the appearance of the connection to the chain.

North Side Hangers—The upstream and downstream hanger chains are unbent, and are in better condition than those of the south end. However, the inner hanger of each chain is dented and scored where it sheared through the bottom strut.

The preceding lacks detail. It truthfully indicates the general conditions, however.

CAUSE OF THE ACCIDENT—THE MOST LIKELY THEORY

Reference to the drawing on p. 523 of this issue or to the slightly less complete drawing p. 423, Aug. 31 issue, shows a cruciform rocker casting between suspended span and lifting girder, by way of which the end of the suspended span rested on the lifting girder. This casting formed a sort of universal joint to support the span; it had a longitudinal pin below and a transverse pin above. Sliding movement along the longitudinal pin was helped by extra lubrication while the span was on falsework, but was blocked by bolting on four retainer plates when the span was floated off on scows.

Study of the sketch of the method of support of the span on the lifting girder here will show that there was a measure of instability in the arrangement, the weight being on top, with a universal joint under it, and the support—the point where the girder was hung to the chains—being practically at the same level as the universal joint. To explain how the truss span comes to be in the river and the lifting girders are still there we must look for some action that would unseat the span. The span might slip off, or the girders might kick out, or the rocker casting between girder and truss might break.

No actions are known to have occurred in the hoisting that would favor either a sliding off of the span or a kicking out of the lifting girder. The most probable explanation at the moment is that the cruciform rocker casting between lifting girder and end of the suspended span broke.

The rocker whose break caused the disaster is believed to be the one at the south upstream corner whose fall was first observed. Fracture of the casting probably occurred on the side facing the center line of the span. At any rate the break must have been so located as to let the truss corner fall on the lifting girder off center and near the midspan edge. Then the girder rotated, kicked back, and let the corner of the span slide off with nothing but the strength and support of the portals and sway bracing to hold it from falling into the water. Clearly the portals and laterals of a 640-ft. span were not equal to such duty.

As soon as the span lost its support at the south upstream corner by breaking of the rocker, the diagonally opposite corner or the north downstream corner probably lifted slightly so that the line of support then was diagonally across the span from south downstream to north upstream corner. Failure of the portals and lateral system and possible rotation of the trusses and failure of the whole structure both downward and sideward was inevitable in this condition.

All this took only four or five seconds, of course, and observation of the individual actions is uncertain. However, it appears that the upstream truss began to crumple as the span tilted, the ends of the truss going down and its center up.

I send you my account after most favorable opportunities to observe the facts and conditions of the hoisting operation, seeing the collapse from an excellent vantage point, and after interviewing a large number of eyewitnesses to check my observation of the facts of the collapse and engineers of the bridge company as well as independent engineers to check the facts as to the behavior of the individual parts.

No one knows at present whether more will be learned concerning this failure in the sequel, as the physical evidence is deep under water. It is only fair, however, to impress on all engineers that the case is so extraordinary that, especially in view of the wonderfully scrupulous work of the Commission and the bridge company, judgment should be formed only tentatively, and final conclusions held in abeyance. One thing only is clear just now, that an awful accident has happened.

What Caused the Quebec Bridge Disaster?

By no means the least remarkable circumstance in connection with the disaster to the great bridge at Quebec is the fact that although the disaster was witnessed by over a hundred of the most eminent structural engineers in both Canada and the United States, including the expert staff engaged in the erection of the bridge, it was not certainly known for 24 hours after the disaster occurred, exactly where was the initial point of failure.

When the first unsatisfactory press dispatches came over the wire, telling practically nothing, save that the span had collapsed and gone to the bottom of the river, the most probable conjecture seemed to be that some-

thing had gone wrong during the ticklish process of mooring the span into exact position and transferring it from the scows which brought it to the bridge site to the hanger chains. As this was apparently the most delicate part of the work, the possibilities of trouble from wind or tidal currents or other unforeseen interference with making the connections to the cantilever arms loomed large in the imagination. These first speculations were of course swept one side when the news came that the span had been actually hung from the lifting chains and hoisted some distance when the collapse occurred.

Many engineers, doubtless, then turned their thoughts to a consideration of a possible failure of the lateral bracing of the span. While every precaution had been taken apparently to make sure that the bridge should be kept level while the lifting process was going on, it was of course possible that one corner might be lifted more rapidly than the others, producing a torsional effect on the truss. It was conceivable, at least, that such uneven hoisting might have gone far enough to throw such stresses into the lateral bracing as to cripple some of the members and cause the collapse of the entire structure. It will be evident that the truss would not have to be cant over very far before its enormous weight would overload the wind bracing.

More circumstantial reports from Quebec disposed also of this hypothesis. Had the failure occurred in such a manner, the great number of expert bridge engineers who watched the failure would have promptly identified and reported its evident cause. The same is true of any failure within the suspended span, from whatever source; and more than this, the span was a piece of bridgework of ordinary type, and internal failure was no more likely than in any other large truss bridge.

The huge lifting chains by which the truss hung suspended did not fail. They are intact, with the lifting girders still attached to their lower ends. The suspended span alone is gone. That is the basal fact. The swinging chairs on which the span was seated are intact—barely scratched—but their burden has slipped off into the river.

The place of initial failure, then, is narrowed down to the place where the truss rested on the girders which hung from the bottom of the lifting chains. Study of the drawings of this connection, on page 523 of this issue, shows that the rocker bearing on which the truss rested was over 4 ft. higher than the pin-bearing through the girder by which the load was transferred to the chains. To hold the girder from rotation, there were plate-and-angle-connections between the top chord of the girder and the short built-hanger link which formed the means of attaching the girder to the chains.

Of course, so long as the span remained absolutely level, and absolutely central on the lifting girder, there was no tendency to tip or rotate the girder, and no strain against the plate-and-angle connection just mentioned. At the moment of the accident something happened which upset this condition of perfect central loading. If the steel casting which supported the south upstream corner of the span broke under its load of some 1,300 tons, its fracture would have produced the loud report with which the disaster began. As the truss shoe settled down through the broken fragments of the casting, it was bound to destroy irrecoverably the delicate condition of central loading. It may have kicked the supporting girder out from beneath it; it may have shoved it far enough to one side so that the unbalanced load on the girder would rotate it on its pin and let this corner of the truss slide off. In either case the wreck of the suspended span would occur as the eye-witness account describes it. One corner unsupported and only the unsteady swinging chairs under the other three, the ponderous span was bound to go to the bottom.

Temporary Trestle to Complete Kansas City Bridge

Pending the construction of the new 23rd Street Trafficway in Kansas City, Mo., a temporary trestle is to be built to connect the Allen Ave. viaduct in that city with the east end of the new Kansas Ave. bridge over the Kaw River in Kansas City, Kan.

The Wyandotte County (Kansas) Commissioners, about two years ago, let a contract for the new Kansas Ave. bridge to replace the old structure which was not only inadequate but also did not conform to the Government and Kaw Valley Drainage Board's requirements of a two-pier structure in the 734 ft. established channel. The new bridge was located to meet the proposed 23rd Street Trafficway Viaduct on the Missouri side and is practically completed at a cost of $480,000. It has two 317-ft. spans in the channel and the total width of 64 ft. provides for one 6-ft. sidewalk on the south side, a 36-ft. roadway and a double street-car track on the north side.

Although an item of $300,000 for the construction of the 23rd Street Trafficway was included in a recent bond issue, voted in Kansas City, Mo., the city has been unable to come to any agreement with the four railroad companies where tracks are crossed by the proposed viaduct as to their share of the expense of construction. Plans and specifications are being prepared by Harrington, Howard & Ash, Consulting Engineers. The structure will be reinforced concrete and will cost approximately $560,000, including a long approach on Liberty St., which will replace the present Allen Ave. viaduct. The city proposes to pay one third of the cost, the street railways company has agreed to pay one-third and it is expected that the steam railroads will agree to contribute the other third. Court proceedings have already been instituted for the widening and regrading of 23rd St. east of Brooks St. where the new viaduct will terminate, the balance of the city bond issue being used to help pay this cost, and it is possible that the other points in controversy will be settled in the next few months.

In the meantime, the temporary timber structure will be built and the new Kansas Ave. bridge thrown open to traffic, so that the old bridge across the Kaw may be removed before it becomes a menace again in the spring high water. The temporary trestle will be 1,056 ft. long and average about 30 ft. high. It will carry a double-track car line, included in a 36-ft. planked roadway, but no sidewalks. The contract price is $17,450. This sum was obtained by "passing the hat" among the industries in the West Bottoms who will be served by its construction, forty of them contributing $5,090 and by the Kansas City Railways Co. who paid $12,500; the balance will be paid by the railroads and the city.

News of the Engineering World

Sewage Disposal at Lynn

An investigation of methods of disposal of the sewage of Lynn, Mass., conducted by the State Department of Health and the Municipal Council of the city, has resulted in the recommendation that the ocean be utilized, the method of dilution being the most efficient of all systems of disposal now in use.

The dry-weather flow in the sewers amounts to about 12,000,000 gal. per day, while the sewerage system is capable of delivering about 81,000,000 gal. per day, including storm water, at the outlet. The wide variation in rates of flow would make it difficult to provide treatment, but consideration was given to methods of chemical precipitation, filtration, activated sludge, screening and disinfection, as well as the recommended method of disposal by dilution. The success of the latter method at Boston is cited by the joint board as an example of the value of that method to coast cities.

Pennsylvania R. R. to Detroit

The plans of the Pennsylvania Railroad Co. to expend $40,000,000 to enter Detroit, Mich., are officially announced. The entire traffic of the city has heretofore been handled by the Michigan Central, Grand Trunk and New York Central lines.

From Toledo, Ohio, the present northern terminus of the Pennsylvania lines, the tracks will be extended a few miles to connect with the Pere Marquette R.R., for the joint use of which the Pennsylvania R.R. has already perfected arrangements. This will give the Pennsylvania an entrance to Detroit, where it is proposed to spend a large sum in paralleling the belt-line railway of the Detroit Terminal R.R., owned jointly by the New York Central and Grand Trunk lines. A part of the project is a large terminal passenger station at Detroit.

The new belt line will connect with practically all the large automobile factories and will give the Pennsylvania a share in an immense traffic.

The Detroit terminal work will be in direct charge of B. V. Sommerville, former Principal Assistant Engineer, Southwest System, Pennsylvania lines west of Pittsburgh. He has offices in Detroit under the title of Resident Engineer.

A Steel Stack Section fell nine stories when one leg of the A-frame of the derrick, with which it was being hoisted, broke. The stack is being erected on the Pittock Block, Portland, Ore., by Clay S. Morse & Co. The last section was being hoisted when the accident occurred. The section is 10½ ft. in diameter by 15 ft. long, and weighs 4,500 lb. The fir boom—109 ft. long by 18 in. in diameter—dropped across the parapet and snapped in two. According to press dispatches the stack section and the wreckage from the derrick struck a roof 100 ft. below, but did not go through. The rivets will be taken out and the section straightened.

Expanding the Corps of Engineers, U. S. A.—The largest number of commissions in the Corps of Engineers, U. S. A., to be awarded to a single class of West Point graduates is the 24 given to the members of this year's class. Of these 24 graduates, 21 are first lieutenants, their commissions dating from July 1. The other three are second lieutenants—the only ones now in that lowest rank. The law effective July 1 provides for 79 second lieutenants. The basic salary of a first lieutenant is $2,000 per annum and of a second lieutenant $1,700.

The Bronx Valley Sewage, of Westchester County, N. Y. has been ordered purified by the Secretary of War. Notice was served on the manager of the Bronx Valley Trunk Sewer Commission, Sept. 11, that the government would bring suit to force the removal of the sewer outfalls in the Hudson River at Yonkers unless the county builds a treatment plant. In 1912 the Federal Government brought a suit against the Bronx Valley Sewer Commissioners, but the action was dismissed upon the signing of a stipulation that the commission would treat the sewage before it reaches the Hudson River. John J. Brown is manager of the Bronx Valley Sewer Commission.

Flood Prevention in Kansas has been investigated by an engineering committee appointed by the Kansas Flood Congress which convened a year ago. The committee states that conservative estimates fix the net losses in Kansas during the past thirteen years at approximately $52,000,000, or an average of $4,000,000 annually. In addition to the direct monetary losses, there has been a large amount of indirect property loss and more than 100 lives have been sacrificed. The committee believes the cost of flood protection must be paid, to a large extent, at least, by taxation upon the citizens of Kansas. Coöperation of the Federal Government is necessary, however, for the complete solution of the problem, as many of the streams which cause damage in Kansas have part of their drainage area beyond the state boundaries. The committee says that the first thing to be done in the study of flood prevention is to establish stream-flow records and it recommends the establishment of a number of river gaging stations. The committee was composed of T. J. Strickler, Chairman, Engineer of the Kansas Public Utilities Commission; H. A. Rice, of the State University Engineering School; H. B. Walker, State Drainage Engineer; A. A. Robinson, formerly of the Santa Fe, and Charles Wenzer, consulting engineer of the Union Pacific. Under a recent act of Congress, a survey is to be made by the U. S. Engineers of the Kansas, Neosho and Cottonwood rivers and their tributaries, with a view to the development of some system of flood control.

PERSONALS

L. P. Matthews, a structural engineer of Lima, Ohio, has joined the staff of A. M. Frink, Architect and Engineer, Cleveland, Ohio.

C. R. Chevalier, formerly Division Engineer of the Boston & Maine R.R., St. Johnsbury, Vt., has been appointed Roadmaster of the Portland Terminal Co., Portland, Me.

Joseph K. Gannett, Assistant Chief Engineer of the Samuel Austin & Son Co., Cleveland, Ohio, has been appointed Chief Engineer of the Philadelphia branch of the company.

C. S. Izant, formerly of Izant & Frink, Architects and Engineers, Cleveland, Ohio, is now in charge of factory output and production methods for the Corning Glass Co., Corning, N. Y.

J. W. Beardsley, M. Am. Soc. C. E., Chief Engineer of the Porto Rican Irrigation Service since 1910, has resigned. He was formerly Director of Public Works of the Philippine Islands.

Robert Lenox Stewart has resigned from his position as Engineer in charge of the New York City office of the Casper Ranger Co., of Holyoke, Mass., to accept a position with the Bridgeport office of the Austin Co.

George J. Percival has resigned his position with Day & Zimmerman, Consulting Engineers, Philadelphia, Penn., to become Engineer of the Philadelphia office of the F. W. Mark Construction Co., of Cleveland, Ohio.

William Ferguson, formerly Supervising Engineer at Detroit, Mich., of the Travelers Insurance Co. and the Travelers Indemnity Co., of Hartford, Conn., has been promoted to be

Traveling Supervising Engineer with headquarters at Hartford.

Robert E. Bell has been appointed Assistant Chief Engineer of the Samuel Austin & Son Co., Cleveland, Ohio, to succeed Joseph K. Gannett, promoted, as noted elsewhere. Mr. Bell was formerly in the office of the Building Commissioner of Cleveland.

J. Albert Holmes, M. Am. Soc. C. E., of Somerville, Mass., has been appointed Resident Engineer for the J. G. White Engineering Corporation on three large earth dams for the Southern Power Co., near Bridgewater, N. C. He was formerly Division Engineer in charge of the construction of the Charles River dam at Boston and Resident Engineer of the Somerset dam at Somerset, Vt.

Lincoln Bush, M. Am. Soc. C. E., Consulting Engineer, New York City, has been selected as the third arbitrator to settle the Galveston (Tex.) causeway controversy. As noted on p. 333, of "Engineering News," Aug. 17, the other members of the board of three outside engineers to investigate designs and report upon the project are Prof. A. N. Talbot, of the University of Illinois, and Prof. George F. Swain, of Harvard University. Professor Talbot represents the steam-railway interests and Professor Swain the county and electric-railway interests. These two selected the third member of the board.

John L. Thompson, M. Am. Soc. M. E., formerly Supervising Engineer of the Travelers Insurance Co. and the Travelers Indemnity Co., of Hartford, Conn., has been made Superintendent in general charge of the engineering and inspection division of the company. He was born in Bridgeport, Conn., and was graduated from Sheffield Scientific School, Yale University, class of 1904. After four years with the New York Telephone Co. as an assistant engineer in the department of building construction, he entered the service of the Travelers Insurance Co. as an inspector. He has served three years in the underwriting department. **William P. Eales,** formerly Supervising Inspector at Philadelphia, Penn., has been made Assistant Superintendent.

Dr. Allan D. Risteen, M. Am. Soc. M. E., recently Engineer of the Travelers Insurance Co. and the Travelers Indemnity Co., of Hartford, Conn., has been appointed Director of Technical Research and Safety Publication Work of the company. He was born in Amesbury, Mass., and was graduated from the Worcester Polytechnic Institute in civil engineering in 1885. Later he took graduate courses at Dartmouth College and Yale University, where he was awarded the degree of Doctor of Philosophy in 1903. For two years after graduating from Worcester Polytechnic Institute he was with the United States Coast and Geodetic Survey. Later he was Editor of "Power." The change has been made in Dr. Risteen's position to enable him to devote more time to technical research and the direction of the publications on safety.

The Corps of Engineers, U. S. A., has been increased by 24 graduates of the 1916 class at West Point. Of these 21 are First Lieutenants at $2,000 per annum, plus allowances, dating from July 1. The 21 most fortunate ones are: John H. Wills, Lynchburg, Va.; Raymond G. Moses, Denver, Colo.; Wilhelm D. Styer, Trenton, N. J.; John W. Fraser, Suffern, N. Y.; Charles H. Cunningham, Lawrenceville, Ill.; Dwight F. Johns, Rockford, Ill.; William A. Snow, Washington, D. C.; Thomas D. Finley, Washington, D. C.; William H. Britton, Cedar Rapids, Iowa; Elroy S. Irvine, Phœnix, Ariz.; Notley Y. Du Hamel, Brooklyn, N. Y.; William H. Henderson, Georgetown, Tex.; Robert G. Guyer, Fort Sill, Okla.; Edward C. Smith, Marion, S. C.; Robert A. Sharrer, Westminster, Md.; Ludson D. Worsham, Evansville, Ind.; Edward G. Bliss, Washington, D. C.; Ralph G. Barrows, Rochester, N. Y.; Holland G. Robb, La Crosse, Wis.; Frederick W. Bonfils, Fort Constitution, N. H., and Robert R. Neyland, Jr., Greenville, Tex. The three commissioned Second Lieutenants are William M. Hoge, Jr., Lexington, Mo.; Stanley L. Scott, Louisville, Ky.; and Tattnall D. Simkins, Albany, Ga.

F. B. H. Paine, William McClellan, F.'s Am. Inst. E. E., and **Horace T. Campion,** M. Am. Soc. C. E., announce the formation of the firm of Paine, McClellan & Campion, of 25 Church St, New York City, and 1420 Chestnut St., Philadelphia, Penn. Mr. Paine is the son of Charles Paine, prominent a few years ago as a railroad engineer and executive. In 1886 he entered the Westinghouse laboratories at Pittsburgh as a student. In 1889 he went into the Southwest as salesman and engineer for the purpose of establishing electric lighting companies in municipalities. From 1891 to 1893 he was in the employ of the Edison General Electric Co., Siemens & Halske Co. In 1893 he returned to the Westinghouse Co. spending most of his time at the New England office, finally becoming its manager. In 1905 he became General Manager of the construction work of the Ontario Power Co. In 1911, he opened an office in New York as a Consulting Engineer. Mr. Campion was born in Philadelphia and has spent the larger part of his engineering life there. After a short time spent in architectural offices he entered the electric railway field through the old Philadelphia companies, finally becoming assistant to the Chief Engineer, William S. Twining. About 1905 he left the Philadelphia Rapid Transit Co. to take charge of the Philadelphia office of the Re-Enforced Cement Construction Co. During this time he had charge of the very heavy reinforced concrete forebay work of the Ontario Power Co. at Niagara Falls. He has made a special study of the design and construction of car-barns and substations. In 1907 he associated himself with William McClellan, another member of the present partnership, in order to carry on a constructing and consulting engineering business. A biographical sketch of Dr. McClellan was published in these columns on Aug. 31, at the time of his appointment as Director of the Wharton School of Finance.

OBITUARY

Emil Herter, M. Am. Soc., M. E., Chief Mechanical Engineer of the Edison Storage Battery Co., West Orange, N. J., died Aug. 24 aged 59 years.

Douglas Boyd Carpenter, formerly a member of the engineering staff of the Southern Ry., has been killed in action in France, where has was serving as Second-Lieutenant of the Royal Engineers of Great Britain.

Henry Horne, for many years an erecting engineer with the Allis-Chalmers Co. of Milwaukee, Wis., died recently at the home of his daughter, in Anniston, Ala. He had charge of the installation of many water-works plants throughout the country.

Walter Chur, President and General Manager of the American Railway Supply Co., of New York City, died Aug. 29, at his home in East Orange, N. J. He was born in Philadelphia 71 years ago. He had been General Manager of the company for 33 years.

Charles W. Kouns, General Manager of the Eastern lines of the Atchison, Topeka & Santa Fe Ry., died at his home in Topeka, Sept. 3. He was born in New Holland, Ohio, Oct. 22, 1854, and began railway work as a brakeman in 1871. He was promoted through the train-dispatching and transportation departments to be General Manager of the Western lines in 1909. He was transferred to the Eastern lines in 1910.

ENGINEERING SOCIETIES

NATIONAL ASSOCIATION OF STATIONARY ENGINEERS.
Sept. 11-16. Convention, Minneapolis, Minn. Secy., Fred W. Raven, 417 S. Dearborn St., Chicago.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 18-20. Convention in Philadelphia, Penn. Asst. Secy., C. D. Fawcett, 29 West 39th St., New York, N. Y.

ASSOCIATION OF IRON AND STEEL ELECTRICAL ENGINEERS.
Sept. 18-22. Convention in Chicago. Secy., W. O. Oschmann, Oliver Iron and Steel Co., Pittsburgh, Penn.

ROADMASTERS AND MAINTENANCE OF WAY ASSOCIATION OF AMERICA.
Sept. 19-22. Convention in New York City. Secy., P. J. McAndrews.

MICHIGAN GAS ASSOCIATION.
Sept. 21-22. Annual meeting in Detroit, Mich. Secy., Clark R. Graves, Lansing, Mich.

AMERICAN PEAT SOCIETY.
Sept. 21-23. Annual meeting in Washington, D. C. Secy., Julius Bordollo, Kingsbridge, N. Y.

AMERICAN CHEMICAL SOCIETY.
Sept. 25-30. Annual meeting in New York City. Secy., Charles L. Parsons, Washington, D. C.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 9-13. Convention in Newark, N. J. Secy., Charles C. Brown, Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 9-13. Convention at Atlantic City, N. J. Secy., E. B. Burritt, 8 West 40th St., New York City.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 17-19. Convention in New Orleans. Secy., C. A. Lichty, C. & N. W. Ry., Chicago.

AMERICAN GAS INSTITUTE.
Oct. 17-20. Annual meeting in Chicago. Secy., G. G. Ramsdell, New York, N. Y.

RAILWAY ELECTRIC SUPPLY MANUFACTURERS' ASSOCIATION.
Oct. 30-Nov. 3. Annual meeting at Chicago. Secy., J. Scribner, General Electric Co., Chicago.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Nov. 14. Convention in Washington, D. C. Secy., W. H. Connolly, Washington.

RAILWAY GARDENING ASSOCIATION.
Dec. 5-7. Annual meeting in New Orleans. Secy., Charles E. Lowe, Sewickly, Penn.

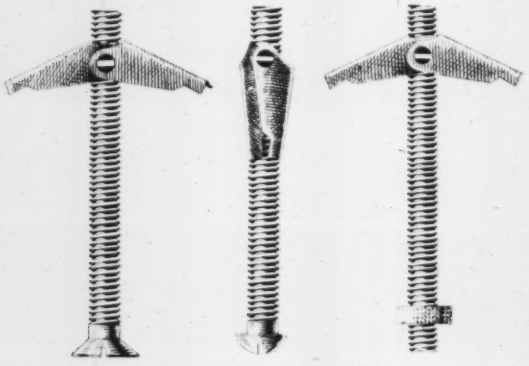
The Association of Port Authorities will hold its fifth annual convention in Montreal, Canada, Sept. 13 to 15.

The Washington State Association of County Commissioners and the Washington State Association of County Engineers will hold a joint convention at Tacoma, Wash., Sept. 14 to 16. A number of technical papers will be presented, as will also a number of addresses on subjects of general interest to highway officials.

Appliances and Materials

An "Automatic" Toggle Bolt

A new design of toggle-bolt head, called the "Arrow," has just been put on the market by the U. S. Expansion Bolt Co., Duane St., New York City. The head is formed by two 1-in. hinged wings and a trunnion nut engaging the bolt threads. A spring inside the wings keeps them opened out except



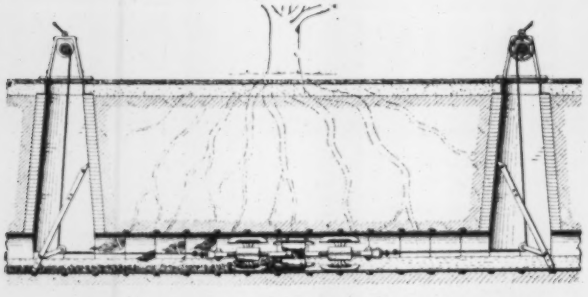
"ARROW" TOGGLE BOLT

when passing through the bolt hole. When the bolt is in place and the wings have been freed the inner edges bear directly on the bolt threads, giving a holding power equal to the strength of the bolt itself and locking the bolt against turning. It is claimed that these hold in corners with a single wing open. Samples are furnished on application.

* * *

Two-Way Sewer-Cleaning Machine

A rotary-blade sewer-cleaning machine and root cutter, which can be pulled in either direction with equal facility, has been introduced by the Buffalo Municipal Equipment Co., Buffalo, N. Y. The device is made to clean pipe sewers from 8-in. diameter up. Sets of equipment may be rented, as well as purchased. It is claimed that roots or any kind of



TWO-WAY SEWER-CLEANING MACHINE AND ROOT CUTTER

obstruction can be removed with this device, which is designed with strength of parts particularly in mind. The manufacturer guarantees to replace any part broken in actual use within five years.

New Noiseless Non-Metallic Gears

A new material for gears, having all the noiseless qualities of older non-metallic products, greater strength and better mechanical characteristics, has been developed by the Westinghouse Electric and Manufacturing Co., of East Pittsburgh. It bears the trade name "Bakelite Micarta—D", and consists of layers of heavy duck bonded together under pressure with "Bakelite"—the synthetic phenol-formaldehyde resin. This gear material can be made in plates of up to 2 in. thickness. It has a tensile strength, parallel to laminations of 10,000 lb. per sq.in.; a compression strength of 30,000 lb. per sq.in. perpendicular to laminations, and of 17,000 lb. per sq.in. parallel to laminations; a traverse strength of 17,000 lb.; a coefficient of expansion per inch per degree Centigrade, of 0.00002 in. in the direction parallel to laminations and of 0.000085 in. in the direction perpendicular to laminations. It has a specific gravity of 1.4, weighs 0.05 lb. per cu.in., and has a water absorption of 0.25 to 2% by weight, depending upon the relative amount of edge surface exposed. Shrinkage and oil absorption is practically zero.

End plates are needed only where the face width of gears is to be more than 2 in. so that two plates have to be riveted together, where the service is extremely severe, or where the diameter is several times the face width. It is claimed that the new gears can in general be substituted for steel, cast-iron or bronze gears and with the same dimensions as those of the old gears.

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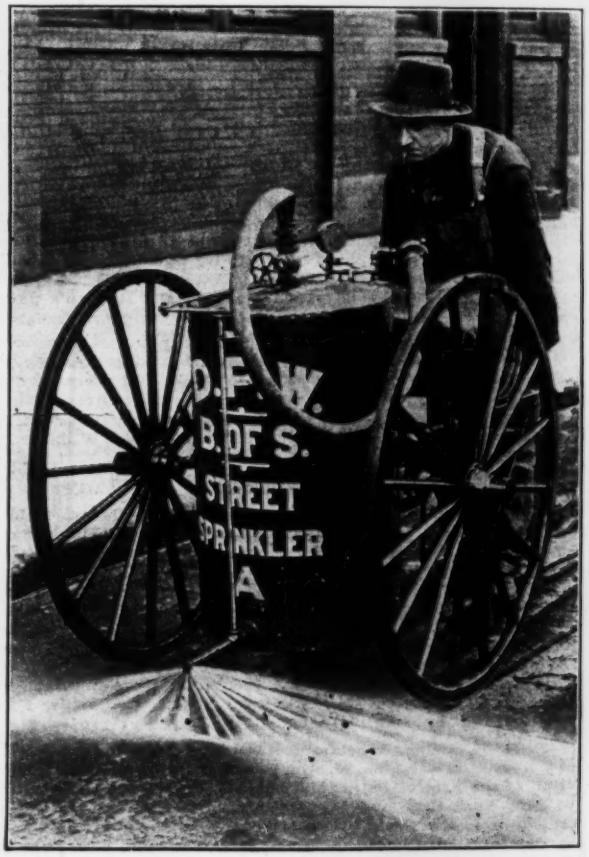
Hand Pressure-Sprinkler

The accompanying view shows a new type of pressure-sprinkler recently placed on the market by the Buffalo Municipal Equipment Co., of Buffalo, N. Y. The water-tank capacity is 45 gal. A small horizontal compressed-air tank behind the water tank furnishes sufficient pressure to throw



HAND SPRINKLER FOR STREETS OR LAWNS

the water 35 ft. laterally, it is claimed. The needle shower is controlled by a lever within reach of the operator. Although this device is designed for street sprinkling in advance of sweepers, it is claimed to be particularly well adapted for sprinkling public halls, playgrounds, tennis courts, lawns, railway-station waiting-rooms, etc.